TRAFFIC COMPACTION

OF

ASPHALT CONCRETE SURFACE COURSES

A thesis presented for
the degree of Doctor of Philosophy
in Civil Engineering
in the University of Canterbury
Christchurch, New Zealand

by

W.D.O. PATERSON

1972

ABSTRACT

The subject of this thesis is the compaction of asphalt concrete surface courses occurring under the action of normal traffic with special reference to the behaviour of the high stability materials commonly used in New Zealand.

A survey is made of the relevant characteristics of asphalt concrete and of the stresses and temperatures which exist in the surface course during trafficking.

A modestly-sized full-scale pavement testing track was designed and constructed. The track had a concrete base and provided control of load, traffic distribution and temperature but not vehicle speed. Two parallel highway studies were conducted.

The rate of compaction was generally rapid initially decreasing to zero at what is defined as a "stable state". The magnitude of the stable state for a given mix was found to depend on load and temperature and the relative effects of these were evaluated in terms of tyre contact pressure and viscosity. None of the properties tested proved to be an absolute indicator of stable state. Binder viscosity and film thickness were found to be predominant factors, layer thickness and maximum aggregate size had an interrelated but secondary role and construction density had a significant effect for light traffic conditions which diminished as the contact pressure and temperature were raised.

Measurements of transient strain profiles were made using induction coil gauges. Typical profiles were obtained and a marked increase in dynamic stiffness was observed for a decrease in layer thickness.

A mechanism of compaction is postulated which emphasises the role of particle orientation.

Present specifications are generally supported by the evidence but a number of particular comments are made.

ACKNOWLEDGEMENTS

During this study many people have given me assistance and I wish to record my gratitude to them.

Professor H.J. Hopkins, Head of the Civil Engineering Department for overall supervision of the project and for personal assistance:

Messrs A. Williman, J.S. Pollard and T.A.H. Dodd, my supervisors, for their indispensable advice, encouragement, assistance and direction:

The many members of the Technical Staff involved for their help and especially Messrs H.H. Crowther and S.R. Robinson for their cheerful and competent approach to arduous work in sometimes difficult conditions:

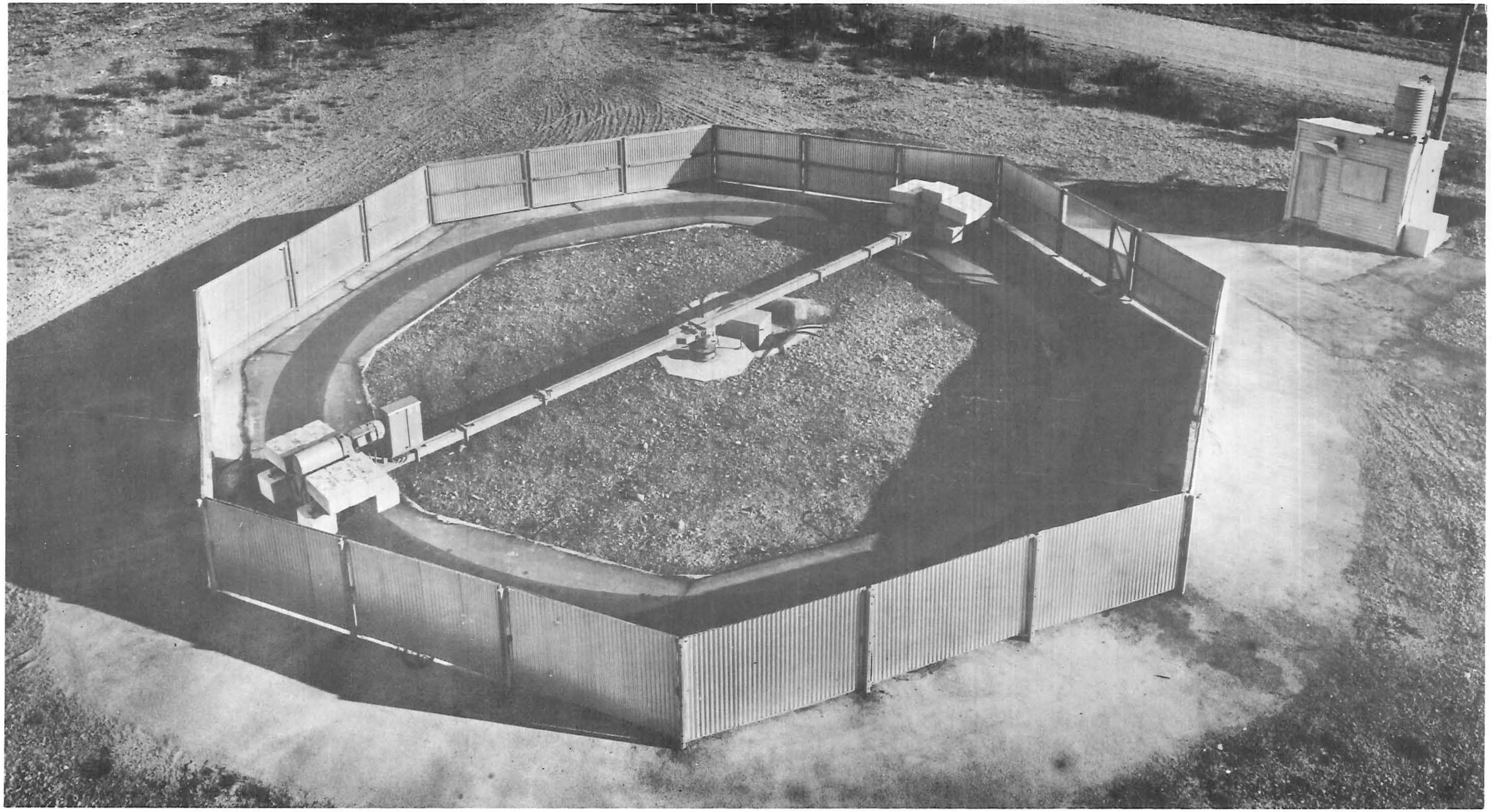
The National Roads Board for a Postgraduate Scholarship in 1968 and for substantial financial contributions to the project:

Many individuals and groups for their contributions to the design and construction of the Testing Track including Roading Division, Head Mechanical Design Office, Southern Plant Zone, Christchurch District, District Laboratory and Residency - all divisions of the Ministry of Works; Dunlop (N.Z.) Ltd and Certified Concrete Ltd:

British Pavements (Canterbury) Ltd for donation of test materials, use of laboratory facilities and technical assistance:

Miss K. Evans, Messrs W. McClelland and H. Patterson, Armstrong & Springhall Ltd, and many others for assistance in preparing this document:

Finally I wish to thank my wife, Ros, for her constant encouragement, help and patience, especially during the lonely hours of Testing Track Widowhood and for her assistance in all weathers.



Frontispiece : UNIVERSITY OF CANTERBURY PAVEMENT TESTING TRACK

FOREWORD

This thesis is published in two volumes. Volume I contains the thesis and references and is essentially self-contained. Volume II contains the Appendices which include supporting information and detailed results.

Units of quantity throughout this thesis are presented according to SI metric standards. Where quoted work was originally in Imperial or other units, the original values and units are included in parentheses.

Superscripts in the text denote quoted references and these are listed at the end of Volume I.

Where they might be confused with a quotation, comments by the author on quoted references are denoted by *( ).

The term "air voids unit" (a.v.u.) is defined as 1% air void content by volume and this avoids misunderstanding when discussing changes in air void content.

VOLUME I

TRAFFIC COMPACTION

OF

ASPHALT CONCRETE SURFACE COURSES

CONTENTS

CONTENTS (cont.)

CONTENTS (cont.) Page

CONTENTS (cont.) Page

VOLUME II

FIGURES

FIGURES (cont.)

TABLES

CHAPTER ONE

TRAFFIC ON ASPHALT PAVEMENTS
- GENERAL CONSIDERATIONS

The phenomenon of traffic compaction is introduced. Fundamental aspects of compaction and some previous studies of the phenomenon are described.

1.1 COMPACTION OF ASPHALT CONCRETE SURFACE COURSES

1.1.1 Introduction

The design, construction and performance of the asphalt concrete surface course of a bituminous pavement has received a lot of attention in overseas research for many years. Most New Zealand practice is based on this overseas knowledge and a dense-graded asphalt concrete is used with moderate confidence throughout the country. There is however, some uncertainty and this is grounded in the question: "How many of the factors implicit in this practice are directly applicable in New Zealand?". One important factor is the amount of densification expected to occur under the action of traffic in the first few years of life of the surface course. This factor is the subject of this study because the mixtures used and the conditions existing here differ in some important aspects to those pertaining overseas.

1.1.2 Compaction in Two Stages

The compaction of the surface course occurs in two stages. The first stage is during construction and probably about 95 to 97 percent of the final density is achieved at this stage. The mix is laid at a temperature of approximately 150^{o}C usually by a paving machine which screeds and tamps the mix into a uniform mat. Then, while the layer is still at temperatures above 80^{o}C, it is thoroughly compacted by rollers, usually including both the steel-wheel and pneumatic-tyred types. The second stage of compaction occurs under normal traffic. Although the tyre pressures and loads may be of the same order during this stage, the pavement temperatures are much lower. During the day, maximum pavement temperatures seldom reach 60^{o}C and generally fall between 25^{o}C and 40^{o}C, and at night they are usually in the range 5^{o}C - 20^{o}C. Hence the rate of densification is slow and as the density increases the mix becomes stronger and offers more resistance to compaction so that the rate decreases gradually to zero. This state many have termed "ultimate density". Since this densifying process takes from two to six years, exposure to the atmosphere during this period may have so hardened the asphalt binder, and thus increased its resistance to compaction, that it never attains the laboratory prediction of ultimate density ("laboratory compacted density").

1.1.2 (cont.)

When the mixture is at its ultimate density there should be preferably some air voids still contained in the mixture. This content of air voids, expressed as a percentage by volume, should preferably fall within two limits, usually 3% and 5%, according to most design codes. This widely accepted philosophy is described, for example, by McLeod in the Asphalt Institute's Mix Design booklet[1] as follows:

> "When the air voids are too low (approaching one per cent or less) the pavement is likely to flush or bleed. Since the error in the air voids determination may be about one per cent due to the limits of precision of currently available test methods, the minimum air voids value specified for paving mixture design should be three per cent, to provide some margin of safety against pavement flushing or bleeding. The maximum air voids value permitted for the design of dense graded paving mixtures should be five per cent. When the air voids value exceeds five per cent, water and air can enter the pavement too easily and cause damage."

In a later publication, McLeod[2] recommended that the rolling of paving mixtures during construction should attempt to reach 100 per cent of the laboratory compacted density in order to derive immediate benefit of design strength and to achieve greater protection against premature binder hardening.

1.2 SOME STUDIES OF THE PROBLEM

1.2.1 Fundamentals of Compaction

The fundamentals of the compaction process as described by Nevitt[3] are summarised before presenting some overseas studies of the problem and outlining an approach for this present project.

Compaction is not merely a densification process bringing particles together and resulting in a higher unit weight. There are many variables involved with a complex interrelationship between them.

1.2.1 (cont.)

Basically however, it is an energy-consuming process in which variable forces act against variable resistances to give a distinctive result.

There are three basic types of compaction: construction compaction, traffic compaction (or "consolidation" by Nevitt's definition), and laboratory compaction (which is intended to simulate either of the first two types).

The forces involved in compaction vary in intensity, duration, continuity, and in rate of application. The direction of the forces may be either vertical or horizontal or both, and in the laboratory situation, the sample may be confined or unconfined. These forces are met by an internal arch resistance or interlock, frictional resistance, viscous or flow resistance and inertia effects. The magnitudes and relative proportions of these factors vary in different mixes and thus affect the result of the applied compacting force.

The structure of a mix must be distinguished from its density. Different methods of compacting a mix will produce different inter-particle structures within the mix. Even when the same density is achieved by different compaction methods the structure and hence the behaviour of the mix will be different in each case. Too often in the past too much importance has been placed on density with little or no regard to structure. The difference in structural behaviour is accentuated by a higher degree of compaction. Compaction by traffic may produce a slight increase in density but produce a large increase in the stability of the mix. The primary reason for this difference in behaviour is the result of particle orientation effects. The compaction process initially squeezes the particles together with only a minor relative relocation. Increased compactive effort can move the particles into new positions and change their relative orientation <u>if</u> the forces are in the right directions and there is sufficient compactive energy. Such a reorientation will greatly increase the structural strength or stability of the mix. It appears that vehicular traffic is capable of this kind of compactive effort.

Nevitt discusses four common and basic types of compaction process in these terms:

1.2.1 (cont.)

(a) Rolling compaction: consists of repetitive low intensity forces, without impact, but with alternating horizontal forces resulting in considerable particle reorientation and excellent compaction. The rate of densification depends on aggregate size (inertia) and dynamic viscous resistance.

(b) Direct compression: requires a high intensity of force (which sometimes causes severe aggregate degradation) and provides negligible particle reorientation. It is generally only effective in a very cohesive mix with little interlocking effect and where flow is predominant.

(c) Impact: causes high stress intensities and possibly some degradation and meets high inertia and flow resistances. There are some lateral stresses which help to reorientate particles. Correlation with traffic action is good at low densities but poor at high densities.

(d) Vibration: gives an excellent reorientation of particles even though it is dissimilar to traffic action. It is probably unsuitable for high viscosity materials where the dynamic flow resistance is high.
*(Author's comment: a high frequency, producing high strain rates, might cause some work-hardening of the binder and consequently increase the viscosity.)

Nevitt concludes by saying that any design procedure must be based on a test simulating the stresses of road compaction conditions rather than on arbitrarily varying the criteria of one standard empirical test.

Bearing these remarks in mind, let us now consider some studies of traffic compaction.

1.2.2 Some Studies of the Problem

The phenomenon of traffic compaction has been studied in numerous projects, but a small selection of these will indicate the

1.2.2 (cont.)

general tenor of the findings.

(a) Nebraska, U.S.A.

Campen et al[4] in Nebraska, U.S.A., studied the effect of five years' traffic on three basic mixes with 19, 16 and 13 mm ($\frac{3}{4}$, $\frac{5}{8}$ and $\frac{1}{2}$ inch) maximum aggregate size respectively. They recorded five-year field densities of the order of the 50-blow Marshall density and noted that this could be rapidly achieved in a few months of hot weather. The initial density did not appear to affect the final density since two pavements initially at 95 and 98 per cent laboratory compacted density both reached 99-100% laboratory compacted density. They also warn that high laboratory densities are not always achieved under intense traffic, citing the case of a pavement which was only at a relative density of 96.6% after three years of heavy traffic (17,000 v.p.d.). *(That was a 19 mm mix with low asphalt content).

(b) Louisiana, U.S.A.

Shah[5] in Louisiana, U.S.A., published the results of a survey of 61 test sections after three years of traffic. The main purpose of the project was to determine optimum construction methods, but leaving that aspect aside, some conclusions were also reached on traffic compaction. He observed that most of the increase in compaction occurs in the first six months of trafficking, and that most of the sections attained 100% of the 75-blow Marshall density. He also observed that strips which had had high intensity compaction during construction suffered the least magnitude of deformity in rutting. It should be added, however, that the trend of his density versus time results showed some perturbing reversals as well as some features incompatible with other findings.

(c) New York State, U.S.A.

A study was made in New York State in the period 1962 to 1967. Graham et al[6] presented interim findings after two years, and Palmer and Thomas[7] reported on the findings after five years.

Graham et al in the first paper attempted to assess the relative influence of various factors on the density of the pavement in its early

1.2.2 (cont.)

life (see subsection 2.1.5). They also analysed data on pavement density variability and concluded that:

> "(1) density does not vary significantly in the longitudinal direction, (2) initial pavement density varies significantly across the lane, being greatest in the centre and least near the edge, (3) traffic increases pavement density and decreases variation, (4) initially 29 per cent of the pavement areas had a density of less than 95 per cent Marshall. After one year of traffic this was reduced to eight per cent and after two years to four per cent, (5) after two years' service, the pavements are smooth-riding with no observable surface defects."

The paper by Palmer and Thomas examines the traffic compaction and also gives a survey of literature. From the curves of relative density versus time they observe that about half the density increase in five years had occurred by the end of the first year, but after five years the densifying trend was still continuing. After five years, the average density increase in the wheel paths was 3.5 per cent and 2.5 per cent between the wheel paths but most of the 47 strips were still below the 50-blow Marshall density taken at the plant. Furthermore, in the range of low to medium traffic volumes, there seemed to be no specific correlation between either traffic volumes or vehicle loads and the increase in density; a rather surprising result. However, on a "Thruway" test section which was heavily trafficked there was a much greater increase in density. Even there however, the increase seemed to be independent of vehicle loads. The findings generally agree that low initial densities preclude the attainment of laboratory density under traffic, but the authors also suggest that the upper limit of five per cent air voids may be unduly restrictive because the low-density pavements did not exhibit excessive deformations.

1.2.2 (cont.)

(d) Texas, U.S.A.

Epps et al[8] studied fifteen sites in Texas. Each was paved with a different mix and each one was compacted in three subsections with low, average and high compactive efforts. Six sets of measurements were made over a period of two years. Initially, air voids contents ranged between 5% and 25% over the fifteen sites, but after two years of traffic all mixes had densified so that the final air voids contents ranged between 2% and 5%. The subsections of each strip, initially compacted to different densities, densified to virtually the same final density so they concluded that the effects of initial density were slight. Final differences between the subsections of a strip were of the order of 2 to 3 air void units (a.v.u.)*. Strangely however the increase in density between the wheel paths was very close to the increase in the wheel paths in many cases, within one or two a.v.u. That this might be due to thermal cycling was discounted by the observation of insignificant density change during untrafficked periods. Some decreases of density were noted of the order of one per cent between one and four months, and some areas between the wheel paths were denser than in the wheel paths. Some influence of traffic volume was noted but these results were scattered probably because the data was not treated in depth. Details of mix design were included in a separate publication which was not to hand and hence the author cannot relate these results to design densities nor to his own research.

(e) Conclusions

One of the most striking differences between these four studies is the "ultimate" density which is attained. The Louisiana densities are all of the order of the 75-blow Marshall density, but in Nebraska and New York the densities are only of the order of the 50-blow Marshall density or less. On the simple assumption that other factors are constant, which they are not, an obvious suggestion for this difference is that temperatures in Louisiana are higher than those in Nebraska and New York. These results will be discussed in retrospect in the final chapter and explanations will be offered for some of the surprising deviations.

1.2.2 (cont.)

This picture then emerges of the compaction occurring under traffic. The pavement densifies rapidly at first, but then at a diminishing rate until it attains an "ultimate" density which may or may not be that predicted by a laboratory compaction test. The ultimate density achieved may depend on the composition of the mix, the thermal environment, and the volume and loads of traffic. The pavement densifies most rapidly in the wheel paths but there is some evidence to suggest that a significant increase in density may occur between the wheel paths.

1.3 APPROACH TO THE PROBLEM

Now that the problem has gained perspective it is necessary to outline an approach to the problem so that some of the details can be determined.

First and foremost it is necessary to understand the behaviour of the material being used and so this is studied in Chapter Two along with the influence that the various components of the mixture have on its overall behaviour. Following this it is necessary to examine the physical context of the surface course in the pavement system so that the nature of both the imposed and the resisting stresses are understood along with any environmental effects that may be relevant, such as temperature. This is done in Chapter Three.

It is only then with this background knowledge that a choice of testing mode can be made and physical tests performed to assess some of the parameters involved.

CHAPTER TWO

THE MATERIAL BEHAVIOUR OF ASPHALT CONCRETE

The influence of various components on the properties of asphalt concrete is discussed. The rheological properties are studied to elucidate the important factors in its response to traffic and this involves the discussion of a number of theories and the laboratory evidence supporting them. Finally, one empirical test, the Marshall test, is assessed in this context.

2.1 THE INFLUENCE OF THE COMPONENTS ON THE PROPERTIES OF ASPHALT CONCRETE

2.1.1 Introduction

The growing use of asphalt concrete as a strong paving material in the 1930s and 1940s caused most research to be directed towards assessing the influence of mix design on its properties. These properties were determined by various empirical tests which in themselves had limited value, but which acted as measures against which the influence of each component could be assessed. More recent trends have been towards determining the absolute properties of a mixture that are necessary for confident design and evaluation.

2.1.2 Binder

The function of the binder is to bond the mix together and in this the principal factors are film thickness and viscosity. The film thickness of the binder for a given aggregate matrix is determined by the binder content and should fall between two limits. Too thin a film lacks both durability and tensile strength[9,10,11]. Also the film thickness must be great enough to develop sufficient surface tension because this determines the cohesion of the mix[12,13]. However the film thickness should not be too great otherwise the intergranular load transfer and hence the strength of the mixture will be reduced[14].

Viscosity, the second principal factor, significantly affects the stability of the mixture. Together with the surface tension determined by the film thickness it influences the cohesion of the mixture. For a given type and gradation of aggregate and a given asphalt content, the mixture strength is solely determined by the absolute asphalt viscosity. This was shown by Welborn[15] who found that such mixtures had equal strengths when tested at equal absolute viscosities regardless of the source and grade of the asphalt.

Binder viscosity also has a major role during the mixing and compacting processes in determining the eventual properties of the mixture. During mixing the viscosity must be low enough so that the asphalt will envelope aggregate particles on contact resulting in an

2.1.2 (cont.)

intimate coating and a uniform dispersion of materials. However if the viscosity is too low the asphalt will act as a lubricant instead of coating the particles. If the temperature associated with the viscosity is too high, the loss of volatiles will cause hardening of the binder[16]. In the compaction process, the binder viscosity must be low enough to permit adjustment and relocation of aggregate particles within the mix structure and yet it must be high enough to provide stability so that the mix will not deform excessively and lose its structure[2].

Clearly, the film thickness and viscosity of the binder are major factors in the compaction of bituminous mixtures by both construction rollers and vehicular traffic.

2.1.3 <u>Aggregate</u>

The effects of aggregate size, shape and surface texture on the properties of bituminous mixtures have received a great deal of attention this century but the findings are not always consistent[17]. The more important and widely supported conclusions are summarised here.

(a) Gradation

An aggregate gradation may be classified in one of three categories: dense, open or uniform (i.e. one-sized). A dense grading generally gives the highest stability although it is widely accepted that satisfactory pavement stability can be achieved from a wide range of aggregate gradations[18,19,20]. This is mainly because internal friction is the major property contributing to stability and it is largely independent of the contact area between particles[20]. This is not saying that all gradations give satisfactory stability because stability seems to be unpredictably affected by some changes in grading[21]. A dense-graded aggregate with its fairly uniform stress distribution is generally most satisfactory, especially from the viewpoint of degradation[22], but a maximum-density gradation is not necessarily desirable[19]. An open gradation has a higher flexibility[23]. The relative effects of the coarse and fine fractions are largely dependent on aggregate shape and surface texture[24]. However an increase in the finer than 0.075 mm fraction (No. 200 sieve) tends to

2.1.3 (cont.)

increase stability and slightly increase flexibility. A slight excess, with respect to a maximum-density gradation, in the fine sand fraction improves workability but causes some reduction in stability[25]. A dense gradation generally requires a higher binder content than an open gradation[26].

(b) Size

The maximum aggregate size has little effect on mixture strength although an increase in maximum aggregate size will increase the material density. An increase in size also reduces the optimum binder content because the surface area to volume ratio of the aggregate is decreased[27]. The skid resistance of a surface with a few large asperities, as from a large maximum size, has a lower skid resistance than a sandpaper-textured surface although it has superior qualities when hydroplaning conditions prevail[28].

(c) Shape

The influence of aggregate shape is usually regarded in terms of natural, or uncrushed, and crushed aggregates. The use of crushed aggregate increases mixture strength although the effect is least pronounced for dense-graded mixtures. In dense-graded mixtures the aggregate shape in the coarse fraction has little influence on stability but in the fine fraction the use of crushed aggregate causes a significant increase in stability[29,30,31]. However, there is some contrary opinion and this last statement may depend on exact gradation and stone type[32]. Aggregate shape has negligible influence on Marshall flow[32,33]. Angular aggregates give skid resistance properties superior to those given by rounded aggregates[28].

2.1.4 Air Voids and Structure

One of the most popular yardsticks for the performance of asphalt concrete has been the air voids content. Mixture design has had the three-fold aim of providing sufficient stability, satisfactory deformability (e.g. Marshall "flow"), and a void content within 3 to 5 percent as mentioned in Chapter One. A high air void content drastically reduces stability and may precipitate premature hardening of the binder if the permeability is high (cf 3.4.3).

2.1.4 (cont.)

Permeability is a function more related to the size and interconnection of voids than the total void content. A low or zero void content provides no space for the binder to expand at higher temperatures and may cause "flushing" and excessive deformations.

The structure of the compacted mix is not a conscious factor in the design but it is a major factor in the performance of the mixture. As mentioned in the fundamentals of compaction the most stable structure has the particles aligned in a roughly similar orientation perpendicular to the compacting stresses. It is the kneading action of pneumatic-tyred rollers that can cause this alignment and it is the similar stresses existing under vehicular pneumatic tyres than results in the phenomenon of traffic compaction[34,35]. One striking demonstration of the effect of structure is the difference in behaviour between a laboratory-compacted Marshall block and a core cut from a pavement of the same mix at the same density (q.v. 2.3.2).

2.1.5 Conclusions

Mixture design plays a determinative role in the performance of bituminous mixtures. Binder content and type must be selected with respect to film thickness, to viscosity, to durability, to the air void content and to strength parameters. A densely graded aggregate with a high proportion of crushed stone especially in the fine fraction will give a high stability. The exact gradation is not critical for stability and hence can be varied to suit binder content, voids, workability and economy. The air void content and particle structure is strongly dependent on the magnitude and type of compaction.

Graham et al[6], op. cit. 1.2.2(c), performed a series of regression analyses on various factors affecting the initial air voids in the pavement. The factors are presented here in decreasing order as their observation of the relative importance of the factors discussed in this section: "(1) asphalt cement content, (2) pavement rebound deflection, (3) aggregate gradation, (4) asphalt cement viscosity, (5) intermediate roller passes, (6) breakdown roller passes, and (7) final roller passes". However the survey presented in 2.1.2 seems to indicate that asphalt viscosity should have higher ranking. It is

2.1.5 (cont.)

significant that all the mix design parameters are more important than the construction rolling parameters, except for the pavement rebound deflection which has an unexpectedly significant effect.

This summary has been compiled with the ensuing study of traffic compaction in mind and is not to be construed as the formula for an ideal mix, an exercise that would involve the consideration of other factors such as flexibility and fatigue.

2.2 THE RHEOLOGICAL BEHAVIOUR OF ASPHALT CONCRETE

2.2.1 Introduction

The scientific study of materials has for many years idealised solids as being elastic and obeying certain laws, and fluids as being viscous and obeying a different set of laws. So it is not easy to characterise the mechanical behaviour of asphalt concrete which is a mixture of discrete solid particles in a viscous fluid. The fluid, asphalt, has a complex behaviour which is part elastic and part viscous although neither part obeys the laws of "ideal" behaviour. The following evaluation of the rheological behaviour of asphalt concrete is necessary to the understanding of its response to traffic stresses.

2.2.2 The Rheology of Bitumen

In a Newtonian fluid viscosity is the constant ratio of shear stress to shear strain and most softer tars and bitumen come in this category. The harder bitumens, and this includes most commercial grades, are non-Newtonian and the rate of shear is not proportional to the shearing stress. At low temperatures and short loading durations the behaviour is elastic and at high temperatures and long loading durations it is mainly viscous[12,36,37].

Mack[38] developed the concept of "relaxation time" in an elasto-plastic theory. At the instant of load application both solids and liquids behave elastically but within a small time interval (the relaxation time, $\bar{t}$) the molecules move to new positions. Here, the relaxation time is the ratio of viscosity (η) to the modulus of rigidity (G).

$$\bar{t} = \eta/G \qquad 2(1)$$

2.2.2 (cont.)

This relaxation time is a measure of the time the plastic strain lags behind the elastic strain. This concept is still present in the "retardation time" of the more sophisticated linear viscoelastic theory which is the closest and most popular workable approximation to real behaviour to date (see section 2.2.6). Suffice to say that if the relaxation time is long, bitumen can exhibit brittleness 'under short loading duration', and the corollary is true, that for short relaxation time and long loading duration considerable flow can occur.

This brief summary of the complex properties of bitumen serves as an introduction to the consideration of five theories on various aspects of the behaviour of asphalt concrete. They are included here because they each contribute in some way to an understanding of the response of this material to traffic action.

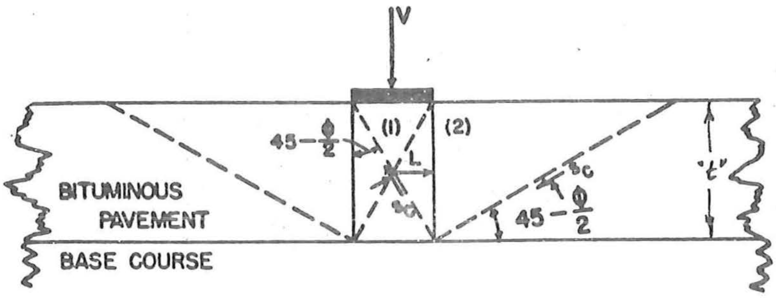
2.2.3 Theory based on Triaxial Parameters

One of the first rational design methods was produced by McLeod (1950)[39] based on the triaxial test parameters of cohesion and angle of internal friction. His concept of bearing capacity was a condition on plastic shear occurring along any possible shear surfaces, and the parameters involved are illustrated in Figure 2.1(a) and summarised in equation 2(2):-

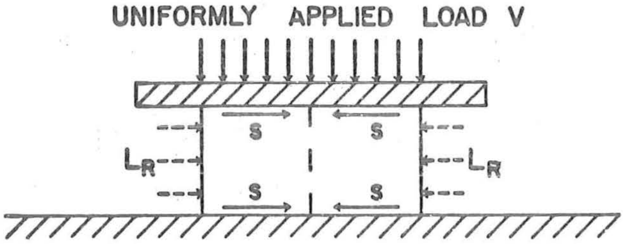
$$V = L \cdot f(c,\varphi) \qquad 2(2)$$

where V = vertical applied stress
L = developed lateral pressure
c = cohesion
φ = angle of internal friction.

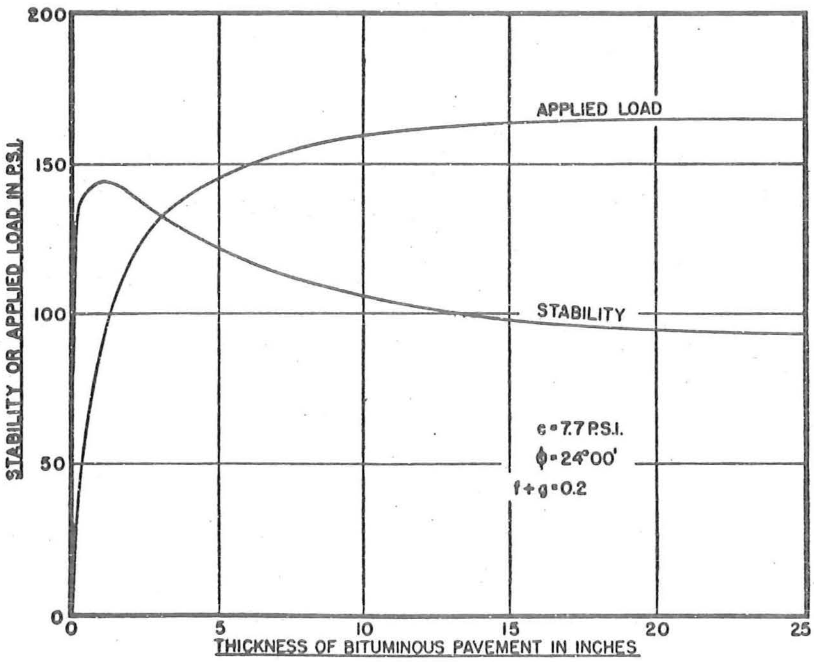
Definite relationships and design curves for these parameters were established, but it was noted that the values of cohesion and internal friction which determine the viscous resistance varied with the rate of deformation. As the rate of deformation increased, the cohesion increased markedly and the angle of internal friction decreased slightly. This resulted in the important observation that strain rates should be selected to best represent the field loading being



(a) Failure Condition



(b) Development of Lateral Support



(c) Effect of Pavement Thickness on Developed Stability

Figure 2.1 MCLEOD'S THEORY BASED ON TRIAXIAL PARAMETERS (from McLeod[39])

2.2.3 (cont.)

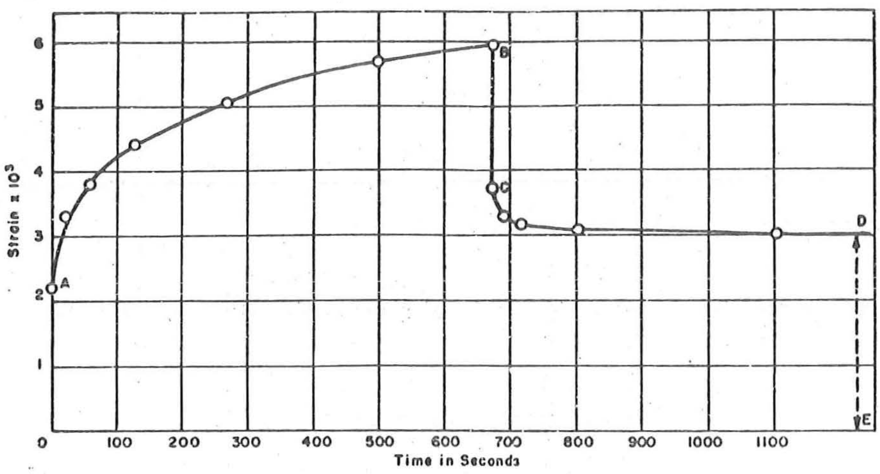
studied. Hewitt and Slate (1967)[40] in a more recent evaluation of the strength characteristics for McLeod's rational design showed that the logarithm of cohesion and of pavement resistance varied directly with the logarithm of asphalt viscosity, while the angle of internal friction is independent of viscosity. They found that cohesion was a function of temperature and the type of asphalt, but that internal friction was independent of them. They also stated that the elastic modulus was a function of temperature, the type of asphalt and the lateral confinement.

One aspect of McLeod's theory received prominent attention in the present study. He demonstrated the influence of boundary friction increasing the effective stability of a pavement element through increasing the developed lateral support by a component L_R (Figure 2.1(b)). Clearly the relative size of this component will be largely influenced by the thickness, t , of the pavement. In a thin pavement the component will be large so that an "unstable" mix might be sufficiently stable in a thin layer. Thus the influence of pavement thickness on the developed stability of any given mix might be similar to that represented in Figure 2.1(c). At great thicknesses, boundary friction has a negligible effect and the developed stability is equal to the mixture stability. However at small thicknesses, the shear strength is supplemented by boundary friction resulting in a more highly developed stability. Superimposing an applied load versus required pavement thickness function, as in Figure 2.1(c), would illustrate the maximum pavement thickness at which the given mix would be suitably stable. This aspect therefore may have significant bearing on the behaviour of the surface course which has a finite and generally small thickness.

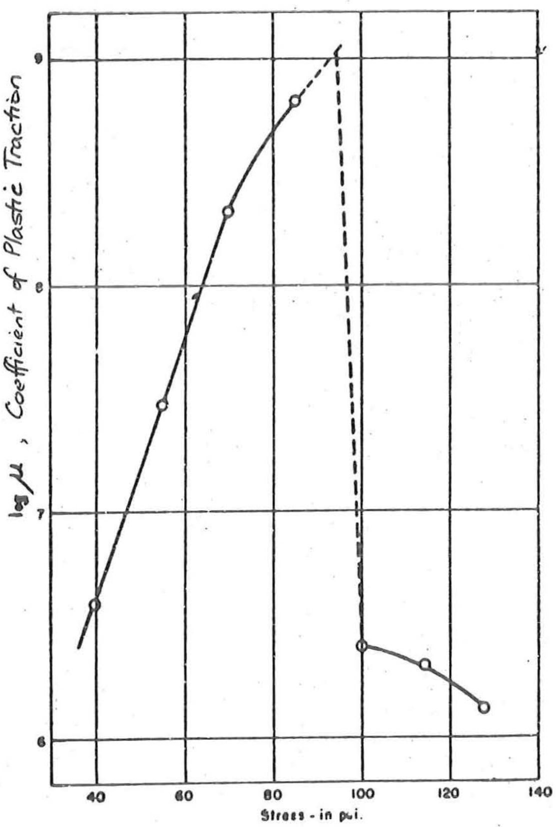
2.2.4 Elasto-plastic Theory

Shortly afterwards, Mack (1954)[38] described an elasto-plastic mechanism to explain the behaviour of bituminous mixtures taking into account the work-hardening which occurs in asphalt under load. His mechanism considers three components of strain shown in Figure 2.2(a) for a sheet asphalt under constant stress at constant temperature:

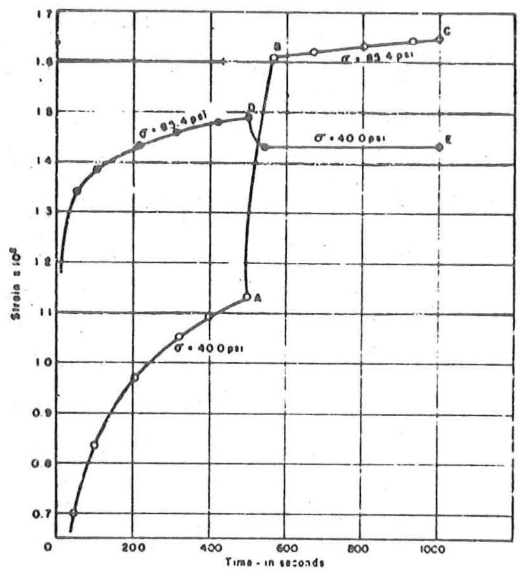
(i) an instantaneous elastic strain (OA) independent of time



(a) Deformation of a Typical Sheet Asphalt under a Constant Stress Step Pulse



(b) Failure in terms of Coefficient of Plastic Traction



(c) Effect of Stress History on Strain Behaviour

Figure 2.2 ELASTOPLASTIC THEORY AND BEHAVIOUR

(from Mack[38])

2.2.4 (cont.)

(ii) a retarded elastic strain which is a function of time

(iii) a plastic strain whose rate decreases with time.

The unloading curve is similar but without the plastic strain: the non-recoverable plastic strain corresponds to the final deformation.

(a) Plastic Deformation

When the plastic deformation is due to plastic flow the stress is a function of strain rate at constant temperature

$$\sigma = (\dot{\epsilon}) \qquad \text{(T constant)} \qquad 2(3)$$

and the strain is a linear function of time at constant stress.

$$\epsilon = \epsilon(t) \qquad (\sigma \text{ constant}) \qquad 2(4)$$

However for bituminous mixtures the strain rate decreases with time at constant stress and temperature and the material hardens. The amount of hardening obtained is the result of work done on the system and this can be either strain - or time-hardening. Experimental evidence shows it to be time-hardening, viz:

$$\sigma = \sigma(\dot{\epsilon}, t) \qquad 2(5)$$

This explicit inclusion of time shows that the history of the mixture dictates its behaviour. For example, if loads of σ and 2σ are applied consecutively each for a constant duration, the total strain is greater than if the loads are applied in the order 2σ, σ, as shown in Figure 2.2(b). The bearing capacity is also likely to be higher.

The condition for failure is defined in terms of the coefficient of plastic traction, μ, where:

$$\mu = \frac{d\sigma}{d\dot{\epsilon}} \qquad 2(6)$$

If the stress on a mixture is gradually increased, the coefficient μ increases to a maximum, at which stage the mixture has had its maximum amount of hardening and acts as a solid body, Figure 2.2(c). Hence the important conclusion is made that the bearing strength of the

2.2.4 (cont.)

mix is not an inherent property but is acquired as a result of work done on the mixture. This conclusion implies, inter alia, that the mixture will fail before it reaches its maximum strength if it is tested at a constant rate of deformation or of strain, as in many of the empirical tests. This also happens if the mixture is tested at constant rate of stress although in this case the bearing capacity is nearer the maximum strength. Furthermore, the compressive strength increases with an increase in a constant rate of deformation or of loading.

(b) Elastic Deformation

The elastic deformation has two components of instantaneous and retarded elastic strain. The latter is an elastic deformation superimposed on a plastic deformation. The mechanism is similar to plastic deformation but it is reversible.

(c) Kinetics and Particle Orientation

One interesting aspect of Mack's paper is his consideration of the kinetics of plastic deformation. He showed that, at failure, the shear and shear stress approach zero as a result of the slip planes rotating, and it follows that there must be an associated change in internal structure. In a compacted bituminous mixture there are a certain number of particles in an "ordered" state, and as these are unlikely to shift under imposed load they are in positions of minimum potential energy. The remainder of the particles are oriented at random in a "disordered" state and are in positions of greater potential energy. From a formulation of the energies involved and cognisance of the stresses acting on an element such as a particle, it follows that, as plastic deformation proceeds and the coefficient of plastic traction increases as a result of work done on the system, the potential energy of the particles must tend to zero. In other words, more particles will assume an "ordered" state as a result of the work done on the system.

This theory reinforces the qualitative assertions on particle orientation and "structure" mentioned in the sections on compaction (1.2.1) and mix design (2.1.4). It seems highly likely that this "re-ordering" or "re-orientating" mechanism is an essential factor in the densification caused by traffic, and particularly in the attainment of a so termed "ultimate density".

2.2.4 (cont.)

(d) Nijboer's Plasticity Theory

Nijboer (1954)[41] described some mechanical properties in the context of his plasticity theory. He found that bituminous mixtures under a short duration of loading generally behaved elastically with full recovery. Under longer durations of load, behaviour was plastic and non-Newtonian. He isolates three components in the resistance of the mix to shear:

(i) interparticle friction - a function of particle shape

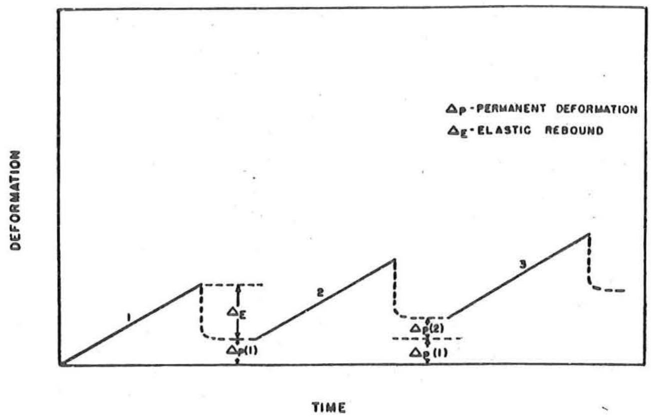
(ii) initial resistance - true particle interlock and the resistance of a thin film of asphalt

(iii) viscous resistance - developed by the rate of shear which is a function of temperature, bitumen hardness, and the filler: bitumen ratio.

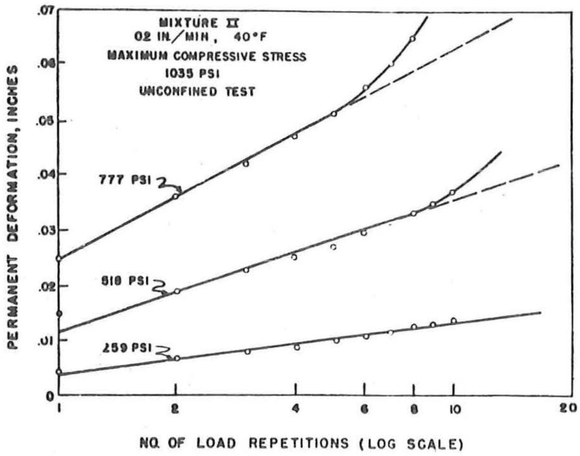
2.2.5 Behaviour under Repetitional Loading

In the same article Nijboer asserts that failure occurs at a higher strength under repeated loading at low stress than it does under repeated loading at high stress.

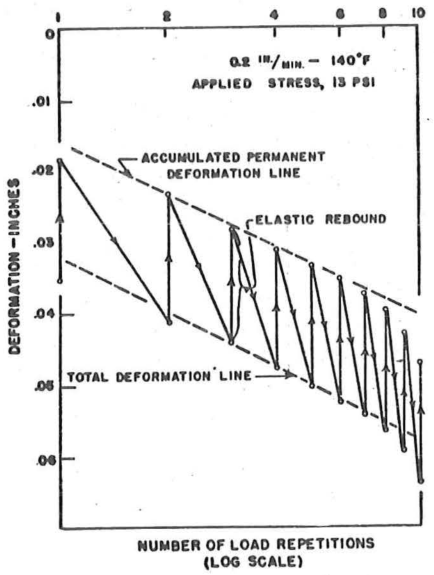
Goetz, et al[42] made a study using a slow cycle repetitive load on unconfined specimens allowing time for most of the retarded rebound to occur after unloading. They showed a linear deformation function against time during the constant stress pulse, contrary to the previous section, and on unloading there was some elastic recovery leaving some permanent plastic deformation, Figure 2.3(a). The elastic deformation per cycle remained constant up to failure but the accumulated permanent deformation increased linearly with the logarithm of the number of load repetitions until failure, when the rate increased rapidly (ib. (b),(c),(d)). As the applied stress was decreased in magnitude, ib. (b), more load cycles were needed to cause this failure and failure occurred at smaller permanent deformations. Under each test condition they found a lower limit of applied stress below which failure could not occur. This "endurance limit", as they termed it, was at approximately 25% of the unconfined compressive strength. It was affected more by temperature than by strain rate and hence they concluded that the endurance limit and the elastic deformation were closely related to film thickness. In discussing



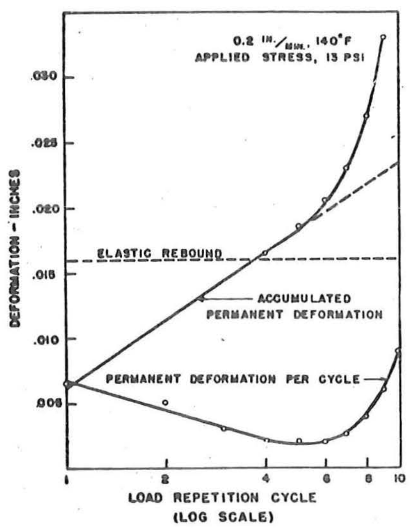
(a) General Relationship Between Deformation and Time for Repeated Load Cycles



(b) Relationship Between Permanent Deformation and Number of Load Repetitions for Various Applied Stresses



(c) Deformation Record of a Specimen Subjected to Repeated Load



(d) Relationship Between Elastic and Permanent Deformation With Repetitive Loading

Figure 2.3 <u>BEHAVIOUR OF UNCONFINED SPECIMENS UNDER REPETITIONAL LOADING</u>
(from Goetz et al[42])

2.2.5 (cont.)

these characteristics they postulated that the elastic deformation occurs in the asphalt film in a thin polymolecular layer firmly bonded to the aggregate particles. Permanent deformations were probably the result of the asphalt film being reduced in thickness in each cycle until a critical thickness was reached. At that point substantial adjustment and reorientation of particles must occur, they postulated, in order to sustain the load. This would give rise to excessive shear deformations and large permanent deformations, a situation which they defined as failure. There are some similarities to this reorientation behaviour in the previous section on Mack's mechanism.

Goetz et al, op.cit., found similar behaviour in repeated load tests on confined specimens. The effect of lateral support had little effect at 4.4°C (40°F) but it increased the maximum compressive stress significantly at higher temperatures, 40-60°C (100-140°F). With regard to thin specimens they found that the lateral support required was equal to or greater than the maximum unconfined compressive strength of the mixture. In other words, this is confirmation of McLeod's hypothesis on the high effective stability of thin layers (cf. 2.2.3).

2.2.6 Linear Viscoelastic Theory

(a) Introduction

The most modern theoretical approach is the theory of linear viscoelasticity which has been under increasing study in the last decade. A viscoelastic material is defined as one which exhibits both elastic and viscous characteristics, with stress related to strain by a function of time in the linear viscoelastic range:

$$\sigma = \epsilon \cdot f(t) \qquad 2(8)$$

Many viscoelastic materials exhibit nonlinear viscoelastic properties, but the theory of linear viscoelasticity is proving to be a very useful approximation for explaining the behaviour of bituminous mixtures. There is a disagreement amongst some investigators in this field on the superiority of this theory over the simpler analyses just discussed including a simple elastic analysis. However, it seems clear that an elastic analysis is only suitable when the conditions

2.2.6 (cont.)

involve low temperature and short duration of loading. Beyond this, it is widely recognised that the rate of loading is a dominant factor in the behaviour of bituminous mixtures, and the linear viscoelasticity theory gives the best handling of it. Pagen[43] concluded that the theory gives a better approximation to real behaviour than elastic theory.

(b) Viscoelastic Models

A number of models comprising linear springs and dashpots representing the elastic and viscous elements have been described[44]. One of the most refined is the generalised Voigt model with an infinite number of such elements.

(c) Concept of the Complex Modulus

The concept of a complex modulus is based on the steady-state dynamic response of a linear viscoelastic material to a sinusoidally applied stress (op.cit.). The response is a sinusoidal strain lagging the stress by a phase angle, φ . The applied stress can be divided into two components, one in-phase and one 90^{o} out-of-phase with the strain. The real part (M_1) of the modulus is then derived from the in-phase component and the imaginary part (M_2) from the out-of-phase component. The general form of the modulus is:

$$M^* = M_1 + iM_2 \qquad 2(9)$$

where M^* is the complex modulus.
The absolute value, $|M^*|$, may be obtained vectorially, and is the ratio of the amplitude of the applied stress to the amplitude of the strain.

Various methods have been employed to find the complex elastic modulus, E^* . However, the work of Papazian[45] and Pagen[43] showed that the dynamic response could be predicted from static loading creep-strain tests by means of a time-temperature superposition principle. This was because the temperature reduction function $f(T)$ and the master creep viscoelastic functions of $E_c(t)$ and $T_c(t)$ could be used to completely define the material at any time and temperature within the tested range. Furthermore, the number of tests required was reduced because the viscoelastic functions could be projected to shorter and longer loading durations and for more intermediate temperatures than

2.2.6 (cont.)

could be normally obtained experimentally. In this way the theory includes consideration of loading time.

Yackovlev and Barenberg (1969)[46] followed this method but found the prediction of $|E^*|$ to be dependent on the initial strain, $\epsilon(0)$. Their work showed $\epsilon(0) = 0$ and hence $|E^*|$ increased linearly with loading frequency. Pagen's work however with $\epsilon(0) > 0$, when similarly treated, gave $|E^*|$ as a constant above a certain minimum frequency. Because the real part of the complex modulus is in effect a measure of viscosity, their work implied a very low elastic response and a very high viscous response. If it is true that the response is mainly viscous, about which there is some doubt, then upper limits need to be placed on E^*. Otherwise absolute values $|E^*|$ greater than the elastic modulus of the aggregate portion of the mix would be possible: an obvious impossibility!

(d) Non-linear Evidence

The behaviour of bituminous mixtures does not always obey the linear viscoelastic laws. Huang[47] found this when testing a sand-asphalt mixture under direct and triaxial stresses. Many others also have noted it, but at present linear viscoelasticity theory gives the best available simulation without too much complexity.

(e) Conclusions

These are some indications of the present problems and the state of the theory of linear viscoelasticity. It is the most powerful tool to date and provides the closest approximation to real behaviour available. The ability to characterise any mixture by only three functions which determine its properties over a wide range has an immediate application in the field of evaluating mixture strengths[43]. Its application to pavement thickness and layer design is only tentative at the moment but is receiving a great deal of attention at recent and proposed conferences.

2.2.7 Conclusions on Rheological Behaviour

Many models and theories have been employed to study the rheological behaviour of bituminous mixtures. They range from a simple elastic analysis (which becomes complicated enough in the analysis of a pavement system), through elasto-plastic analysis, to viscoelastic

2.2.7 (cont.)

theory. This last theory, however, is still at a stage which would involve the entire concentration of a project to relate it to a study such as the phenomenon of traffic compaction rather than to be used as a comparatively quick theoretical evaluation of field tests.

The typical response of a bituminous mixture to loads may now be summarised, considering a repeated constant stress pulse for limited duration and with adequate pauses:

(i) An initial elastic strain will probably occur as soon as the load is applied. This probably occurs in a polymoecular layer firmly bonded to the aggregate particles.

(ii) This will be followed by a part elastic, part viscous deformation at a rate decreasing with time. Probably this is the result of flow in the binder films surrounding the aggregate and the decreasing rate is a function of time-hardening.

(iii) Upon unloading, some deformation is immediately recovered, there is a retarded recovery, and the remainder non-recoverable plastic strain is the permanent deformation. The permanent deformation is probably a result of excessive shear stresses around aggregate particles causing re-orientation.

(iv) The amount of particle re-orientation is a function of the work done on the system. Formulation of the energies involved suggest that the particles should gradually attain an "ordered" state with each in a similar orientation.

(v) The thickness of a pavement layer may have a significant effect because boundary friction might enhance the effective stability of a mix.

(vi) Under repetitive loading, the elastic deformation will possibly remain constant while the permanent deformation increases linearly with the logarithm of repetitions until failure when it increases rapidly.

(vii) At lower applied stresses, more load cycles are required to cause failure, and failure occurs at smaller permanent strains.

(viii) The loading history of the mixture affects its behaviour and the maximum bearing capacity and permanent strain are obtained if the loads are applied in the order of increasing magnitude.

2.2.7 (cont.)

(ix) The bearing strength of a mixture is not an inherent property but is developed as a result of work done on the mixture.

(x) The response will be largely elastic at low temperatures and short loading durations, and mainly viscous at high temperatures and long loading durations.

2.3 EVALUATION OF THE MARSHALL LABORATORY TEST

2.3.1 Introduction

With the knowledge of the characteristics and behaviour of asphalt concrete now outlined it is possible to evaluate some of the empirical laboratory tests which are widely used and so determine what properties these tests actually measure. Only the Marshall test is discussed here since it was to be used in the experimental programme.

The Marshall breaking test is an empirical test which has been related by study and experience to field performance. It is empirical because the compaction of the specimen is by impact rather than a kneading-type action skin to field rolling, it is a constant strain rate test, the sample is partially confined by the breaking heads, and it provides only a one spot value giving no absolute indication of how the properties change with temperature, strain rate or stress rate. These limitations have stimulated a great deal of research into this test method (i) to determine how adequately it does predict pavement performance, (ii) to isolate the sources of variations, (iii) to determine its repeatability, and (iv) to evaluate the measured properties theoretically and experimentally and thus relate them to absolute properties of the mixture.

2.3.2 Correlation with Field Performance

Some early studies revealed conflicting evidence on the correlation between Marshall Test properties and pavement performance, although many of these studies did not consider all the important factors. For example, Dillard[49] found a very poor correlation between air voids in an in-service pavement and the air voids in the Marshall Test. However he did not appear to have considered the variations in mix design nor the relative traffic volumes nor the thermal conditions nor the initial construction conditions. There

2.3.2 (cont.)

are some reports of low stability mixes performing satisfactorily, and some of high stability mixes performing unsatisfactorily[50,51]. Sufficient criteria for satisfactory mix design were eventually derived empirically as the result of a wide range of investigations. As well as stability, these criteria included aggregate voids, mixture voids and "flow"[52]. However the two values of stability and flow were given no specific interaction when being applied to a mixture. Such an interaction was proposed by Metcalfe[53] who incorporated both stability and flow values in a term "Bearing Capacity" by the approximate equation:

$$\text{Bearing Capacity} = 1.55\,\frac{\text{Stability}}{\text{Flow}}\left(\frac{305 - \text{Flow}}{100}\right) \qquad 2(10)$$

where the units are: Bearing Capacity, kN/m^2
Stability, N
Flow, 0.1 mm

This showed a good correlation with field performance. It is noted in passing that the above equation is based on a number of assumptions that are not all applicable over a wide range of values. However it does incorporate some correlation with triaxial strength parameters and is thus a more realistic guide to performance than stability and flow values alone.

It has been mentioned (cf. 2.1.4) that cores cut from the pavement have different properties from blocks compacted in the laboratory to the same density. Some writers[3,54] including the author (q.v. 6.2.2) have noted that core samples have considerably lower Marshall stabilities than the laboratory-compacted blocks. The author's hypothesis is that this is due to the difference in inter-particle structure between the two types of specimen. In the core specimen the particles are likely to be significantly aligned in a near horizontal position due to the kneading action of construction rolling[34,35]. The author has quantitative evidence of this (q.v. 6.8). The stability of such an aligned structure will be high in the direction perpendicular to the direction of alignment but the stability will be low parallel to the direction of alignment. In other words the stability of the core is high in the axial direction as loaded by traffic action but it is low in the radial direction as

2.3.2 (cont.)

loaded in the Marshall breaking test. The impact compaction of the Marshall laboratory block however causes wedging between aggregate particles and little degree of alignment in any direction. Such an interlocking structure will clearly produce a higher stability when tested in the radial direction than the aligned structure of the field core.

2.3.3 <u>Repeatability and Sources of Variation</u>

For any test method it is necessary to know its repeatability and to understand any inherent sources of variation. Vokac[55] made a thorough study of the repeatability of the Marshall test considering twenty sources of variation, fourteen of which were controlled and six varied to assess their significance. These were factors of equipment and time being studied and not factors of mix composition. He found that while a few factors such as the particular hammer used or the order in which the specimens were moulded and broken may be significant, the inherent variability in the Marshall Test itself is low. He obtained a standard deviation of 6.5% from the mean stability and emphasized that deviations greater than 8% were highly unlikely to be attributable to the test itself. Lehmann and Adam[56] found that an experienced operator and use of standardised techniques was necessary to achieve a standard deviation of 7.3%. Temperature conditions for mixing, compacting and breaking were the paramount source of error.

Some of the other sources of variation in stability were investigated by Nevitt[57]. Compaction technique was shown to have a very significant effect with the Marshall impact compaction producing higher stability than vibration compaction to the same density. The postulated cause of this was that the impact compaction caused wedging effects and possible aggregation degradation. This confirms the author's hypothesis mooted in the previous section. Kneading compaction was found to produce stabilities intermediate to impact and vibrating methods. The maximum aggregate size has a significant effect on specimen uniformity and in a small sample the size of a Marshall specimen the number of 19 mm particles in a 19 mm gradation will have a significant effect on the stability. 16 mm seems to be

2.3.3 (cont.)

the maximum particle size desirable for a 100 mm diameter Marshall specimen. The effect of specimen size and particle size will greatly effect the stress-strain relationships developed to resist deformation. End effects, arch action and a variation in shear resistance across the failure plane are also influenced by the particle : specimen size ratio.

2.3.4 <u>The Properties Measured</u>

With these assessments of the Marshall Test against performance, repeatability and sources of variation, let us now consider what absolute properties the test is actually measuring. Goetz and McLaughlin[58,59] experimentally compared the test with triaxial and unconfined compression tests. They concluded that it is a type of confined test with the confinement arising from the curved shape of the testing heads. Metcalf[53] theoretically analysed the stresses in the Marshall Test as if it were unconfined and showed that the "stability" should be about ten times the unconfined compressive stress. However quoted stabilities are much higher than this and so he concluded that the test was of the "confined" type because the specimen has a low height to diameter ratio and the failure planes intersect the testing head. Estimating the degree of lateral confinement from Mohr theory in parameters of cohesion and internal friction, he then deduced a relationship for bearing capacity as a function of stability, flow and internal friction which then simplified to the relationship given earlier in equation 2(10). From these and other studies[60,61] it appears that the Marshall Test is largely influenced by cohesion and that it is a partially confined test.

Krokosky and Chen[62] made an experimental viscoelastic analysis of the Marshall Test to evaluate what the test reveals about the viscoelastic mixture properties. They found that the stress-relaxation curves at 43°C for various asphalt contents were parallel and separated only by a vertical factor which was a simple non-time-dependent constant equal to the Marshall stability : flow ratio at 60°C. This vertical separation is due to the stiffening action of the aggregate in the asphalt/aggregate mixture and the parallelism of the

2.3.4 (cont.)

curves indicates that the superior performance of high stability mixes is a result of their higher absolute modulus and not of a different relaxation function. The time/temperature superposition principle was proved to be valid, and the stability:flow ratio at 60°C was said to be indicative of the stress-relaxation functions below 43°C because of the high rate of load application in the Marshall Test. They also showed, though not exhaustively, that the Marshall stress-relaxation data may be correlated with the true mixture stress-relaxation functions by means of a correction or shape factor.

2.3.5 Conclusion

The Marshall Test may adequately predict the field performance of a mixture only if the aggregate voids, mixture voids, stability and flow are all considered. Marshall stability is not a direct measure of the stability of the mix in the field due to differences in interparticle structure. Basically, the test is repeatable to a high degree but it is sensitive to variations in technique, temperature and mixture. Other principal sources of variation include method of compaction and dimensional ratios of specimen to particle size. The test is a partially-confined type compression test on a specimen of low height:diameter ratio such that the failure planes intersect the testing heads. It is largely influenced by cohesion. For a particular aggregate gradation, the viscoelastic stress-relaxation functions below 43°C of the Marshall Test are parallel for different asphalt contents and are related by the simple vertical shift factor of the stability:flow ratio at 60°C. These functions may be related to the actual mixture functions by a shape factor.

CHAPTER THREE

THE SURFACE COURSE IN THE PAVEMENT SYSTEM

The interaction of vehicle and pavement is studied to determine the nature of traffic loading. The structural role of the surface course and the contribution of the remainder of the pavement system is analysed. Finally, environmental conditions such as temperature and oxidation are considered.

3.1 INTRODUCTION

By virtue of being the uppermost layer in a pavement, the surface course comes into direct contact with vehicle tyres and the environment from above and gains most of its structural support from the underlying pavement layers. These three factors of imposed load, environmental conditions and structural contribution are the external influences on the behaviour of the surface course, as distinct from the influence of its material composition.

The vehicle wheel load is transmitted to the pavement only over the contact area between tyre and pavement surface whence the pavement distributes it over a larger area to the subgrade. For the surface course, the size and shape of the contact area and the distribution of stresses are expected to be more important than the magnitude of the load.

The "flexibility" of the underlying pavement layer determines to a large extent the flexural stresses and strains which occur in the surface course. When a material as stiff as asphalt concrete is used, the surface course itself plays an important role in stiffening the pavement.

The stiffness of asphalt concrete, in turn, is largely influenced by environmental conditions such as temperature, which affects the asphalt viscosity, and oxidation. Other factors to be considered include surface debris and moisture.

3.2 LOADING CONDITIONS

3.2.1 Factors

The concentration of imposed loads at any location on the pavement surface will be determined by the lateral distribution of the placement of wheel loads.

In the consideration of traffic causing compaction of the surface course, the author surmised that the vertical stresses would be more significant than the magnitude of the wheel load. Hence the size and shape of the contact area and the distribution of pressure over it are discussed. Horizontal stresses caused by vehicles accelerating or decelerating may be severe.

3.2.1 (cont.)

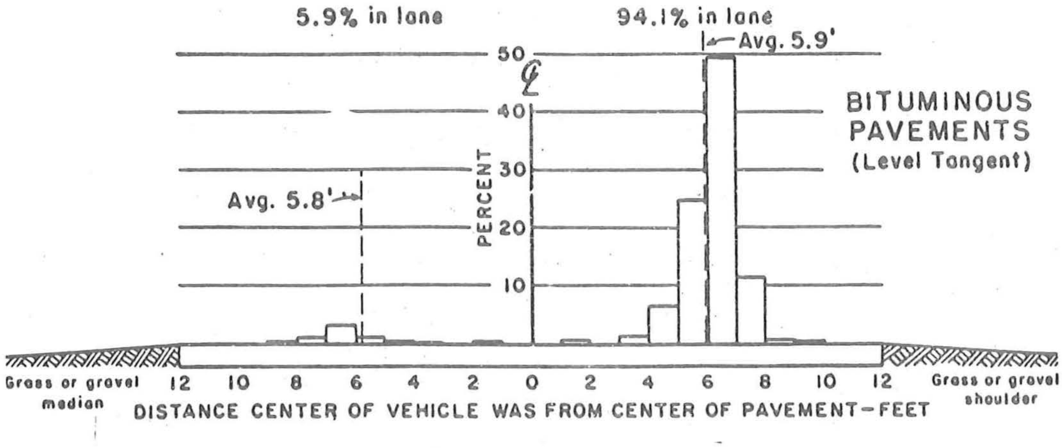
The influence of flexural compression, the surface course being the "upper fibre" of the pavement, might also contribute to compaction but it is considered in more detail in section 3.3. The action of rigid rollers is also considered since these are often used during construction of asphalt concrete pavements.

3.2.2 Lateral Distribution of Wheel Loads

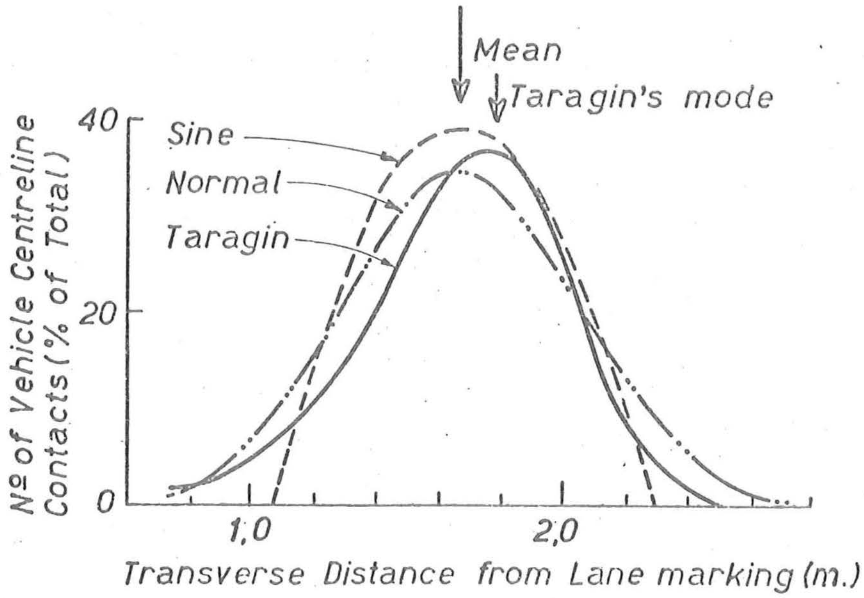
The channelling of traffic on highways, and particularly laning, causes a high concentration of wheel loads on each side of the centreline and aptly termed "wheelpaths". The distribution of individual wheelpaths at any transverse section of the highway will determine the intensity and spread of the loading on the wheel-path area.

Taragin[63] conducted a study in the United States limited mainly to commercial vehicles on the basis that these constituted the heavier more critical loads. The main import of his work is illustrated in Figure 3.1 which shows the distribution of the lateral placement of vehicle centre-lines for laned divided highways. The average occurs 0.06 to 0.24 m (0.2 to 0.8 ft) from the lane centre-line towards the outer edge of the pavement although the mode generally occurs slightly on the other side of the centre-line. In low volumes of traffic the average tends to be further from the centre-line at 0.12 to 0.24 m (0.4 to 0.8 ft). There was some dependence on whether the traffic was free-moving, meeting opposing traffic or travelling adjacent to a stream of traffic. However, the overall distribution shown in Figure 3.1 is a good general indication and the distribution for equal traffic volumes ($\equiv$ area under the curve) seems closer to a sinusoidal than to a normal distribution. The average width of a commercial vehicle is approximately 1.8 m so the average wheelpaths could be expected to be 0.9 m on either side of the distribution's average.

In New Zealand, Niven[64] observed the wheelpaths to be centred 0.9 m (3 ft) in from the centre-line and the edge for a two-lane highway.



(a) Histogram for 4-laned divided highways
(from Taragin[63])



(b) Comparison with mathematical
distributions

Figure 3.1 TRANSVERSE DISTRIBUTION OF VEHICLE PLACEMENT

3.2.3 Tyre-Pavement Interaction

A survey of existing literature shows a comprehensive state of knowledge of the tyre-pavement interaction. The nature of the interaction will depend on: (i) whether the load is transmitted by pneumatic tyre or by rigid roller, (ii) whether the surface is yielding or non-yielding, (iii) whether the load is stationary or moving.

(a) Stationary Pneumatic Tyre on Unyielding Soil

Interest in the topic dates back to the 1930's to Martin[65] and Teller and Buchanan[66] who found that the pressure under a pneumatic tyre was about one third that under a solid tyre for the same load. Many other studies have been made on this topic[67,68,69].

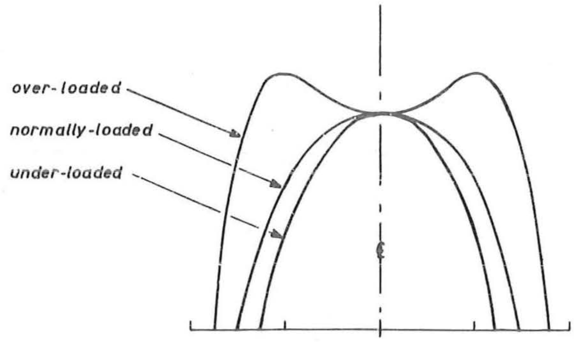
Typical normal pressure distributions are shown in Figure 3.2(a) for the cases of underloading, normal loading and overloading. Large tyre deformation due to overloading (or under-inflation) results in peak stresses occurring near the periphery of the contact area where the flexural deformation of the tyre is greatest. With normal loading, the transverse normal pressure distribution is parabolic, and with underloading (or over inflation) the central peak sharpens. The longitudinal distribution remains relatively uniform with slight peaks at each end.

The studies have shown that the pressure at the centre of the contact area generally exceeds the inflation pressure because of the load carried by the tyre carcass. As a generalisation, Freitag and Green[68] made the following conclusions about the inter-relationship of load, inflation pressure and contact pressure.

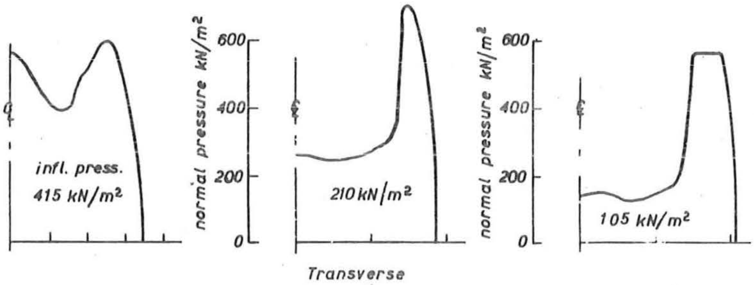
(i) The stress at the centre of the contact area is a function of inflation pressure and is independent of load.

(ii) The maximum stresses occurring at the periphery of the contact area are a function of load and independent of inflation pressure.

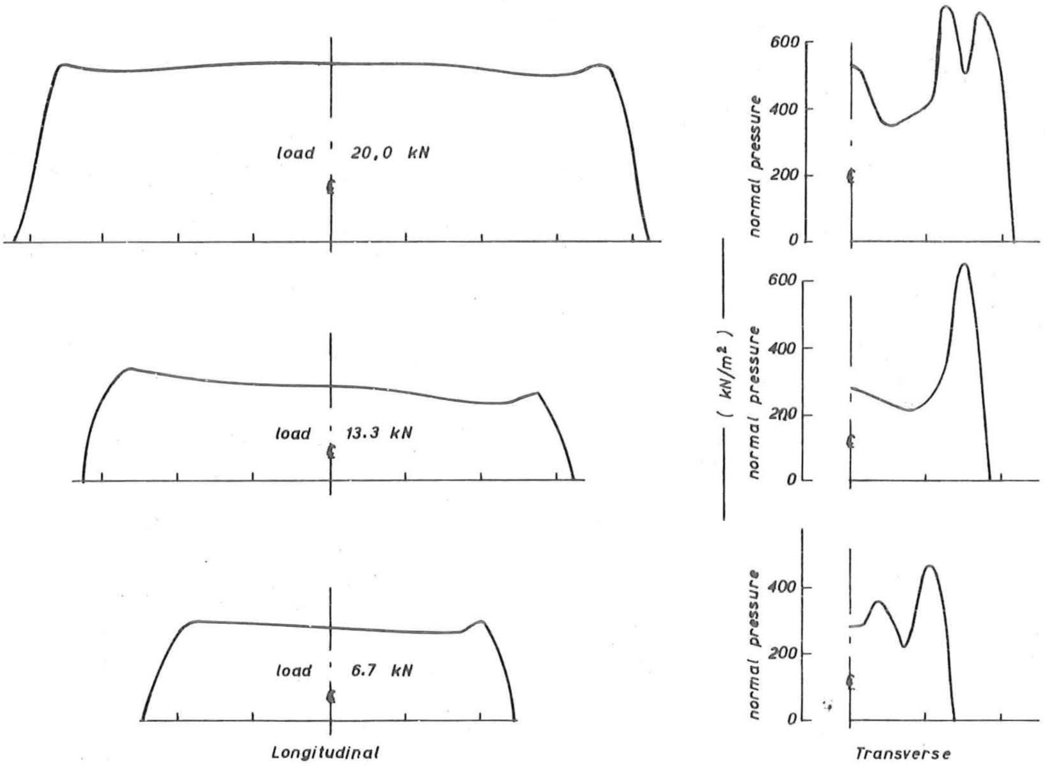
Their results are illustrated in Figure 3.2(b), (c), for a conventional 11.00-20, 12-PR smooth tyre. When the load is constant, the pressure at the centre of the contact area is consistently about 30% higher than the inflation pressure. However, the average contact pressure is

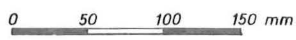


(a) Typical transverse distribution of normal pressure



(b) The effect of inflation pressure on normal pressure distribution for an 11.00-20, 12 ply smooth tyre, 13,3 kN load (after Freitag & Green)





(c) The effect of load on normal pressure distribution for an 11.00 - 20, 12 ply smooth tyre, 207 kN/m² inflation (after Freitag & Green)

Figure 3.2 DISTRIBUTION OF TYRE CONTACT PRESSURE

3.2.3 (cont.)

higher than inflation pressure at 105 and 210 kN/m^2 (15 and 30 psi) and lower at 415 kN/m^2 (60 psi) inflation. The pressure under the sidewalls remains almost constant at about 620 kN/m^2 (90 psi) for constant load but increases as the load increases.

The load carried by the carcass of the tyre varies with tyre size and design and for a given tyre varies with deformation. The flexibility of the carcass depends on the number of plies and a stiffer tyre with more plies distributes its load less evenly, with peaks under the sidewalls, than does a more flexible tyre. The construction also affects the pressure distribution. For constant load and constant tyre deflection, a radial ply tyre has slightly higher contact pressure over a slightly smaller area than does a conventionally constructed tyre, op.cit.

Most of the measurements so far described have been on tyres with the tread buffed off to give a smooth surface. The effect of the tread pattern is to increase the contact pressure by 15-20% due to the reduction in contact area[67]. Irregularities in the tread rubber created during moulding or due to wear can cause departures from the pressure distribution. However, with these adjustments, the pressure distribution under each section of the tread and the envelope of the curve as a whole are comparable to that under a smooth tyre.

In addition to normal stresses, there are also horizontal shear stresses acting over the contact area[69]. Under vertical load the tyre is flattened and its curved surface becomes plane. The flexing of the tyre carcass produces tension on the inner side and compression on the outer side of the carcass. This creates horizontal shear stresses directed towards the centre of the contact area, larger near the centre and decreasing to zero at the periphery. By way of contrast, the shear stresses under a solid tyre are directed outwards. Because the road surface is relatively rigid, the strains produced in the pavement will be negligible but the strains produced in the tyre tread are severe thus creating large slip zones in the area of contact. According to Markwick and Starks[67], these shear stresses can be as high as 480 kN/m^2 (70 psi) on dry roads and 280 kN/m^2 (40 psi) on wet roads.

3.2.3 (cont.)

Very high local intensities of normal pressure can occur on sharp asperities in the pavement surface. These intense pressures are a function of rubber hardness and the shape of the asperity. They are practically independent of tyre size and inflation pressure. They may also cause localised variations in compaction and might possibly influence local cracking in a pavement.

(b) Moving Pneumatic Tyres on Unyielding Soils

The effect of motion on the distribution of normal contact pressure and the contact area is slight. Bonse and Kühn[70] showed that the longitudinal distribution of normal pressure is slightly increased at the leading edge by approximately 2 to 3% but there is no significant effect on the transverse distribution. The contact area tends to be slightly longer than in the static case by up to 8%[71]. The horizontal shear stress distribution becomes unsymmetrical causing a net force equal to the rolling resistance.

Bonse and Kühn made a detailed study of the dynamic forces under vehicle tyres using a small stud embedded in the pavement and fitted with condensers and springs measuring in both horizontal and vertical directions. Peak vertical forces are not significantly affected by speed at zero acceleration or indeed at low rates of acceleration. However, in deceleration there is an increase in the vertical force under the rear wheel despite reduction in load on the rear axle due to the dynamic shift forward to the front axle. This is due to the distortion of the tyre arising from the high torque imposed on it. The longitudinal horizontal forces however are greatly affected by acceleration and the force is approximately proportional to the acceleration. At constant speed longitudinal forces are very small, equal to the rolling resistance, though at high speeds there is a small increase in peak horizontal force at the leading edge. Transverse forces are always directed toward the centre-line of the contact area with negligible changes due to speed or acceleration.

Uncontrolled factors such as dynamic load variation arising from road surface irregularities and structural variability due to the non-uniformity of road materials can cause stress variations of up to $\pm$ 35% in the same nominal construction[72].

3.2.3 (cont.)

Thus normal stresses and transverse horizontal stresses are hardly influenced by speed or acceleration but longitudinal horizontal stresses may be greatly influenced.

(c) Pneumatic Tyres on Yielding Soils

The tyre-pavement interaction in the case of yielding soils depends on whether or not the soil is cohesive. On a noncohesive soil such as sand the shape of the stress distribution is related to the magnitude of the maximum in-soil deflection of the moving tyre, as measured at the centre-line of the tyre cross-section, and sharp peaks occur[73]. However in a cohesive soil such as clay the interface stresses are more evenly distributed and the peak stresses slightly less.

When the wheel is powered the stresses in the "bow wave" are greater and the centroid of vertical stresses is further forward. A towed wheel tends to produce more pronounced peaks than a powered wheel in similar conditions.

(d) Rigid Roller on Yielding Soil

Yandell[74], in model studies using clay and also in a mathematical simulation, demonstrated the residual stresses occurring in an elastoplastic soil under repeated passes by a rigid roller. This situation may be related to the compaction of hot asphalt concrete. For similar loads, the roller induces more intense stresses, especially forward of the load, than does the pneumatic tyre. The maximum residual horizontal stress builds up near the surface under a rigid roller while under a pneumatic tyre it occurs well below the surface leaving the surface relatively undisturbed. Hence, permanent horizontal deflections are greater at the surface under a rigid roller thus illustrating the forward permanent shear which occurs in yielding soils. It is interesting that the rigid roller has more resistance to rolling than the pneumatic tyre. Vertical residual stresses (2 kN/m^2) and deformations are very small and do not differ significantly between roller and tyre.

3.3 STRUCTURAL ROLE OF THE SURFACE COURSE

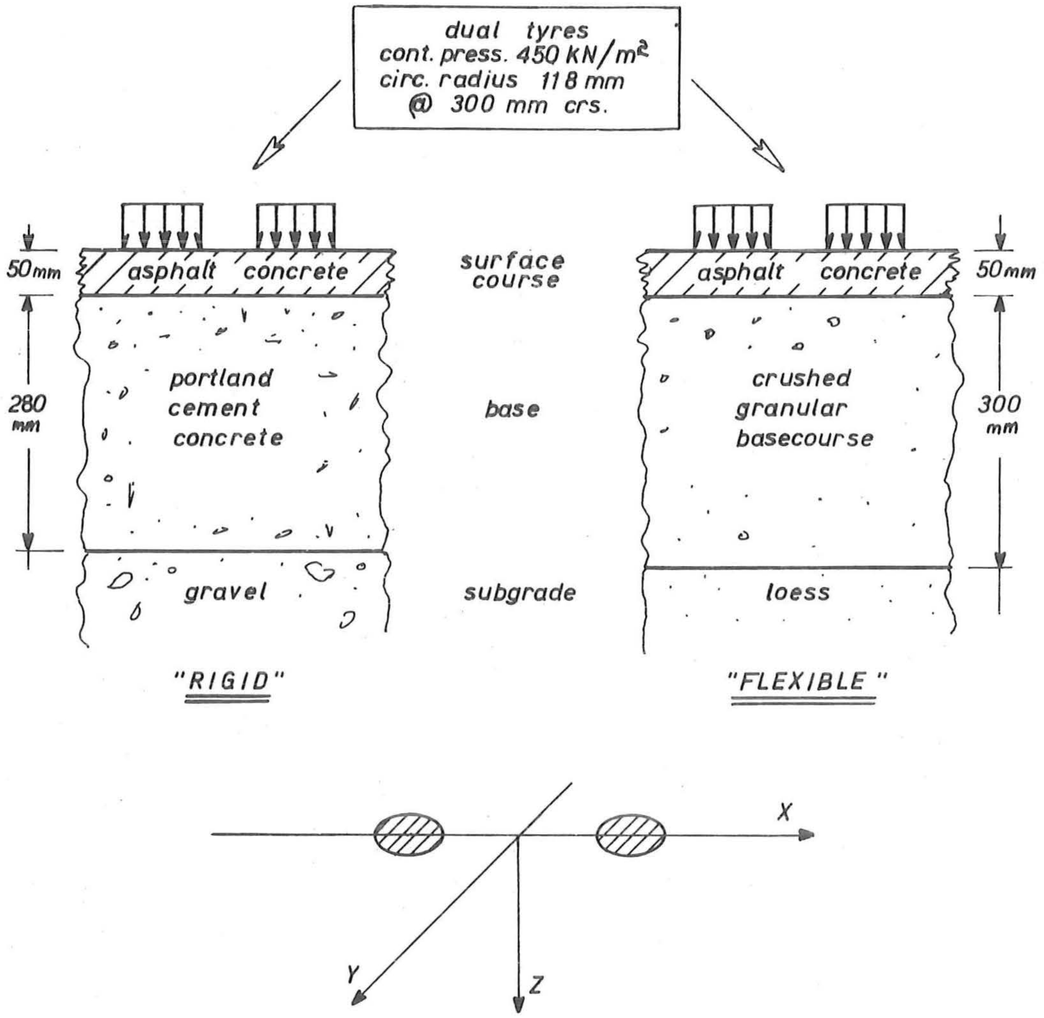
3.3.1 Introduction

Having examined the nature of the vehicular loading on the surface course it now becomes necessary to study the role of the surface course as the uppermost layer in the pavement and the restraints placed upon it by the underlying pavement structure. The understanding required for this study is twofold. Firstly, what is the relative effect on the surface course of the flexibility of the base layers? (This question becomes particularly pertinent later in the design of a testing track). Secondly, what is the stress and strain behaviour of the surface course under the influence of a moving wheel load?

3.3.2 Elastic Analysis of the Effect of Foundation flexibility

Assessing the effect of base flexibility on the behaviour of a surface course is a complex problem. Some observational studies such as those in Chapter One have touched on it empirically; for example, Graham et al[6] who ranked foundation flexibility as second in importance only to asphalt content. The most practicable theoretical analysis at the moment for this problem is the three-layer elastic analysis developed by Jones et al from Burmister's original work and available in the form of a computer program produced by the Shell Co. Ltd[76]. The program enables stresses, strains and deflections to be computed at any point in a layered system subjected to any number of surface loads. It is used here to compare the effects of a "rigid" concrete base and a "flexible" granular base on the behaviour of an asphalt concrete surface course. The choice of material properties is difficult but in this case the properties were chosen to be sufficiently representative and give an adequate contrast between the two situations. The material properties and the conditions are shown in Figure 3.3. This analysis may be compared with those by Lister and Jones[77] and Heukelom and Klomp[78].

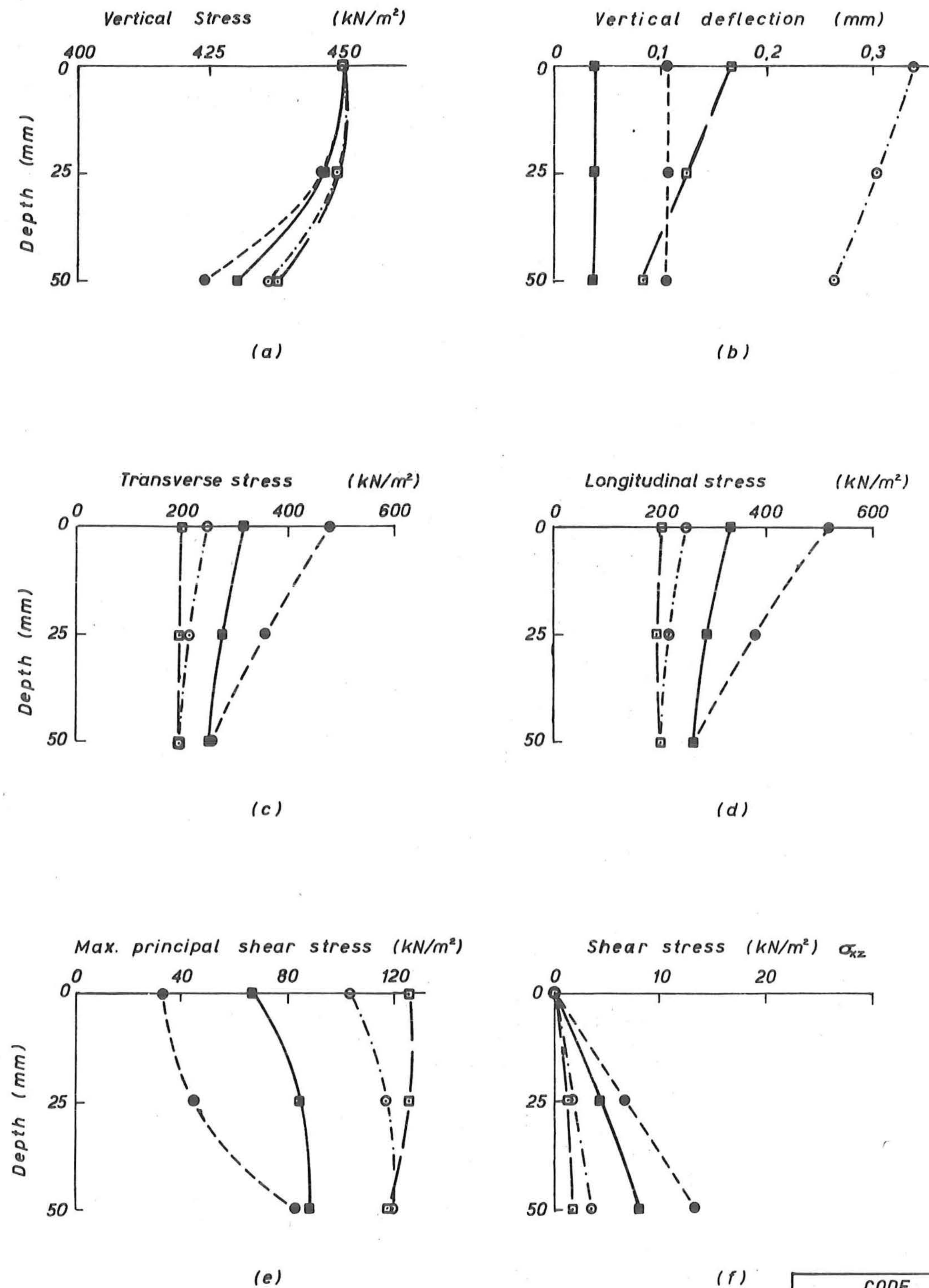
The stress analyses for each situation are shown in Figure 3.4. The vertical stress is modified only very slightly: at 25°C the "flexible" situation has a stress only 1.4% less than the "rigid" situation at the bottom of the surface course and less than 0.5% different at mid-depth. The differences are even smaller at 40°C.



MATERIAL PROPERTIES

Material	Temperature (°C)	Elastic Modulus (10^6 kN/m²)	Poisson's Ratio
asphalt concrete	25	6,0	0,30
" "	40	0,2	0,30
p.c. concrete	both	20,0	0,20
cr. granular b/c	both	7,0	0,35
gravel subgrade	both	0,7	0,35
loess subgrade	both	0,2	0,40

Figure 3.3 CONDITIONS FOR ELASTIC ANALYSIS



CODE		
	Base	Temp.
■	Rigid	25°C
▣	Rigid	40°C
●	Flexible	25°C
⊙	Flexible	40°C

Fig. 3.4 <u>STRESSES COMPUTED BY ELASTIC ANALYSIS</u>

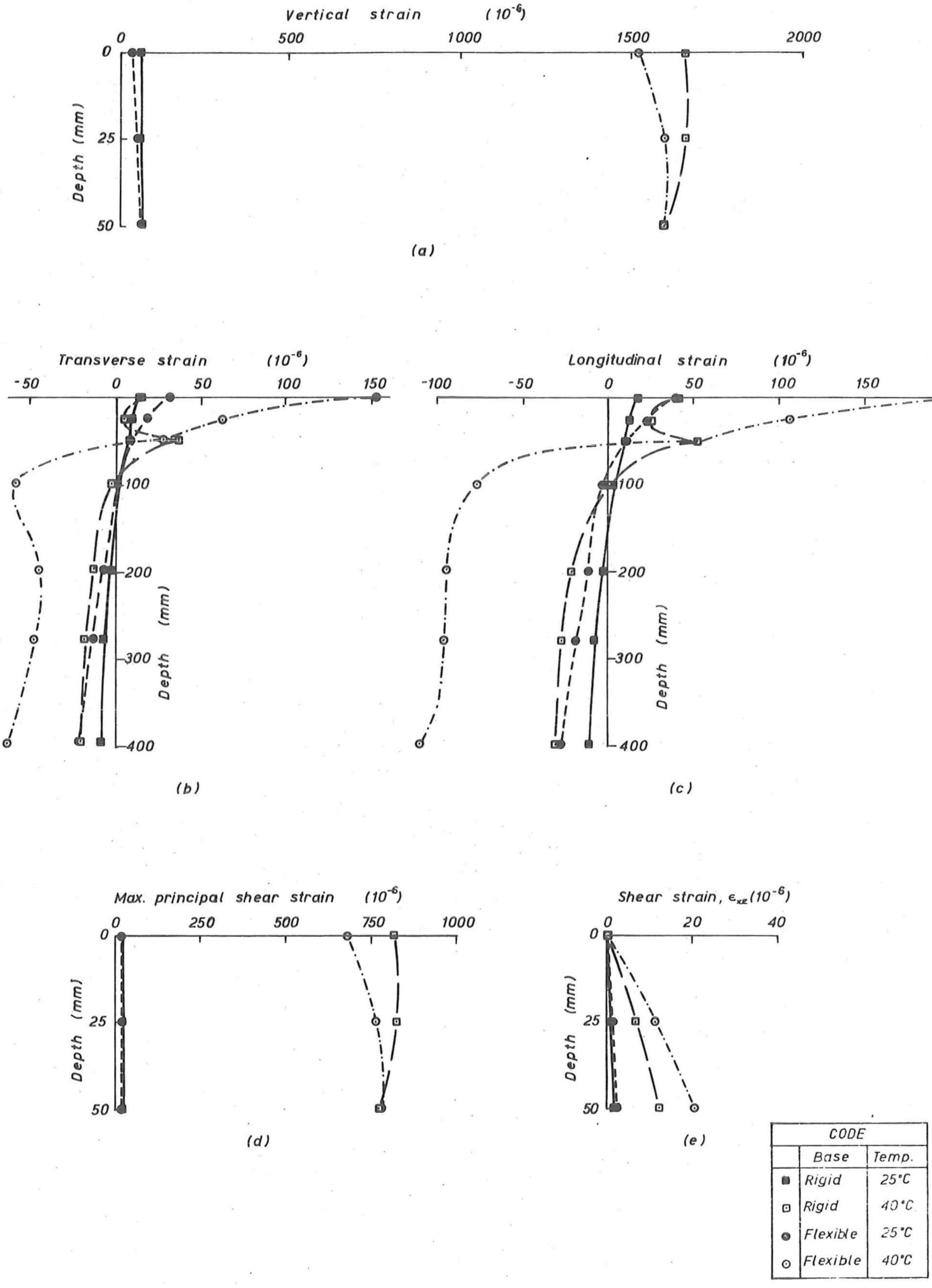


Fig. 3.5 STRAINS COMPUTED BY ELASTIC ANALYSIS

3.3.2 (cont.)

The horizontal stresses, σ_{xx} and σ_{yy} are up to 50% greater in the "flexible" situation although the difference decreases with depth to near zero, and decreases greatly with an increase in temperature. The horizontal shear stresses are negligible.

However, the effects on strain shown in Figure 3.5 are more significant. The effect on vertical strain is very small at 25°C and is still small for the much higher strain at 40°C, 10% less in the "flexible" situation at the surface and 0% at the bottom of the surface course. But the horizontal strains are about 150% higher at the surface and 25°C for the "flexible" than for the "rigid" situation. At 40°C the difference is still greater and the horizontal strain profiles are different shapes. This particular part of the analysis is extremely sensitive to the value of Poisson's ratio (if $\nu = 0.35$, the surface and mid-depth strains for the "rigid" situation become tensile). In other words, this analysis indicates that a "squeezing" action, and possibly instability, might occur in the "rigid" situation if the Poisson's ratio increases much with temperature. Later tests did not indicate such instability and the author has thus used $\nu = 0.30$.

In summary, it appears that the vertical stresses are only slightly lower with a flexible rather than a rigid base but in the horizontal plane the behaviour is considerably affected. While the magnitude of the stresses and strains at high temperatures in the horizontal plane are much smaller than those in the vertical plane, at lower temperatures the magnitudes are comparable.

3.3.3 Dynamic Behaviour of Bituminous Surface Courses

Some investigators have recently been attempting to measure the strain response of a bituminous surface course to an applied load in order to assess the accuracy of theoretical predictions.

Andersson[79] at Stockholm found that the typical compressive strain profile under a moving wheel load had an initial rate of compression of the order of 300% per second beginning at the leading edge of the tyre/pavement contact area. This compression levelled off to a peak under the trailing edge of the tyre contact area, followed by a period of sudden recovery decreasing gradually in rate

3.3.3 (cont.)

to near zero compression. He found that an increase in vehicle speed linearly increases the initial rate of vertical compression but decreased the peak vertical compressive strain to a level which was approximately constant at speeds above 30 km/h. The latter observation is qualitatively confirmed by Klomp and Niesmann[80] and Gusfeldt and Dempwolff[81] who measured horizontal strains at layer interfaces.

Andersson also found that the wheel load had negligible effect on the initial rate of compression but that increasing the wheel load from 0 to 20 kN (2000 kg) caused a slight increase in peak compressive strain (from 10 to 15 microns deflection) and a slight increase in contact time (from 18 to 26 milliseconds). From these relationships, he demonstrated a more marked dependence of peak compression on contact time. Gusfeldt and Dempwolff in their study found that horizontal strain increased with increased load and that the loading period for the transverse gauge was about three times the period for the longitudinal gauge.

The rate of compression "recovery" also increased with wheel speed according to Andersson. Some comparative laboratory studies showed that this was due to the inherent rheological properties of the binder rather than the dynamic components of deformation.

A number of other investigators have used circular plate loading for laboratory studies[82,83].

3.4 ENVIRONMENTAL CONDITIONS

3.4.1 Factors

Environmental conditions play a significant role in influencing the material properties particularly of the surface course and to a lesser extent the other pavement layers. Most important for an asphalt pavement is temperature because this controls the asphalt viscosity and hence the stress-deformation properties. Oxidation of the asphalt binder is important because it increases the binder viscosity. The general topic of ageing includes fatigue considerations as well. Moisture is a secondary consideration for asphalt pavements although it may have significant influence on any underlying granular

3.4.1 (cont.)

layers. Dust or oil lying on the surface of an asphalt surface course may affect the material properties of the surface skin but this is unlikely to be significant unless the deposits are excessive or the asphalt mixture is open graded.

3.4.2 Pavement Temperature

Because the pavement temperature is such an important variable for asphalt pavements, it is necessary both to know the conditions pertaining in the field and to maintain close control of them during research. The road surface temperature is influenced by solar radiation, free and forced convection and conduction. Solar radiation is a function of sunniness, heat and reflectiveness of the environment and it is very low at night. Convection is a function of the relative air and pavement temperatures, the nature of the surface and wind. Conduction effects are mainly a function of the more stable subgrade temperatures. Cf. Appendix II for a theoretical treatment of these in connection with the Pavement Testing Track design.

Dunstan[84] made a survey of temperatures in a bituminous pavement near Melbourne which has a similar latitude ($37^{\circ}40'$S) to that of Auckland ($36^{\circ}50'$S). Some of his results are reproduced here in Figure 3.6. A maximum peak surface temperature of 64.3°C was recorded one day although it exceeded 60°C for only 3% of two months in the summer season, i.e. for 0.5% of the year. The percentages quoted are calculated from the time spent in the particular temperature range as a percentage of total time and thus 3% of 2 months is equivalent to 33 hours duration over the 2 month period. He also quotes maximum peak surface temperatures of 61°C for Maryland, U.S.A. and 54°C for Durban, South Africa. Summarising his conclusions:

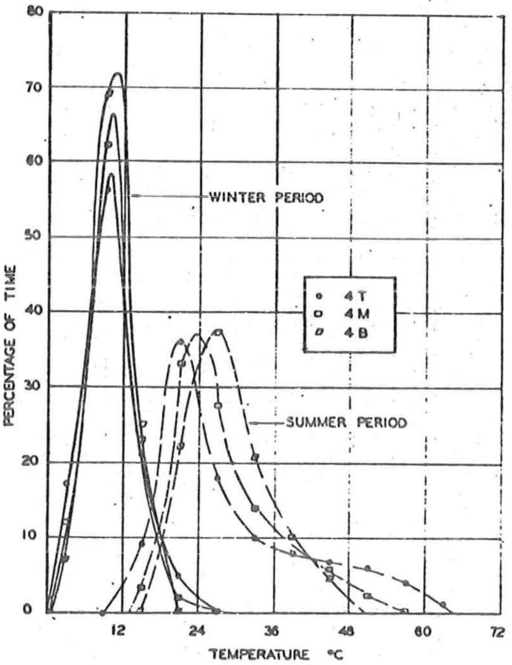
(i) Most of the time the surface was colder than the bottom of the surface course, for a 100 mm thickness.

(ii) Temperatures at a depth of 100 mm appear to be independent of base type.

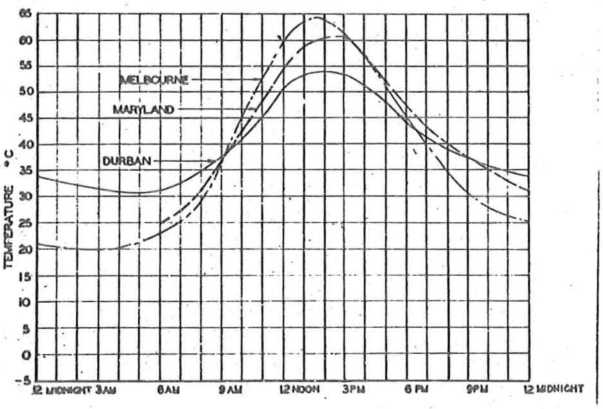
(iii) Maximum temperatures suggested for design use in Melbourne were 60°C for 0–4 mm depth and 50°C for 40–100 mm depth.

(a) Comparison of Summer and Winter Surface Temperatures (^{o}C)

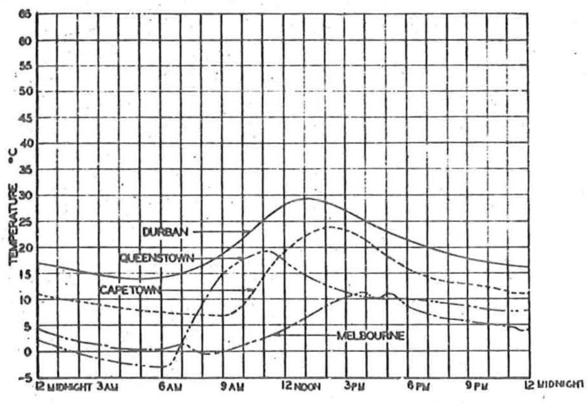
	Summer	Winter	Difference
Mid interval value of minimum temperature	17.5	6.5	11,
Mean modal temperature	21.0	9.0	12
Mid interval value of maximum temperature	48.4	21.5	27
Temperature range	53°C.	30°C.	23°C.



(b) Temperature Variations with Depth for Summer and Winter Periods



(c) Day of Highest Surface Temperature



(d) Day of Lowest Surface Temperature

Figure 3.6 <u>PAVEMENT TEMPERATURES FOR MELBOURNE</u>

(from Dunstan[84])

3.4.2 (cont.)

(iv) The maximum rate of temperature change occurs at the surface and was measured at 8 C°/hour.

(v) The maximum temperature gradient under maximum surface temperature conditions was approximately 0.2 C°/mm for 0–50 mm depth, 0.16 C°/mm for 50–100 mm depth and approximately 0.05 C°/mm for depths below 100 mm.

(vi) A minimum temperature of +5°C seems realistic for design.

Kallas[85] described a year's study of temperatures in 150 and 300 mm thick bituminous pavements in Maryland, U.S.A. The variation of temperature with depth on the hottest day and the duration of temperature at various levels over one year are shown in Figure 3.7. Surface course temperatures exceeding 50°C in the top 100 mm occurred during less than 3% of the year and rarely topped 60°C – a less critical condition than in Melbourne. Galloway[86] shows that the surface course temperature at 40 mm depth in England exceeds 40°C for only 1.6% of the year.

Higher temperatures however are frequently recorded in more tropical areas. Arena[87] recorded high temperatures in southern Louisiana. There the surface course temperature at 10 mm depth exceeded 50°C for the equivalent of 12% of the year, exceeded 60°C for 6% of May through August (2% of the year), and exceeded 65°C for 17% of the time in July, 1962. At the same time the maximum monthly peak at a depth of 50 mm was 55°C. Roberts and Russam[88] quote maximum surface temperatures for bituminous pavements of 61°C in Hong Kong, 42°C in Durban, 40°C in London, 49°C in New Delhi, and 56°C in Kenya. Diurnal temperature variations can be as high as 56°C in Nigeria, 52°C in Kenya, 50°C in Cape Town, 41°C in Ontario and 38°C in England.

Little comprehensive work has been done in New Zealand. Pollard[89] measured 54°C on an old chipseal surface in Christchurch, Niven[64] measured 47°C at 40 mm depth in asphalt concrete at a site near Christchurch, and McNamara[90] measured 37°C surface temperature of asphalt concrete on an exposed elevated site in Auckland. The author during the course of his research in Christchurch has observed asphalt concrete surface temperatures exceeding 60°C on only one or two days in sheltered conditions. The temperatures of the

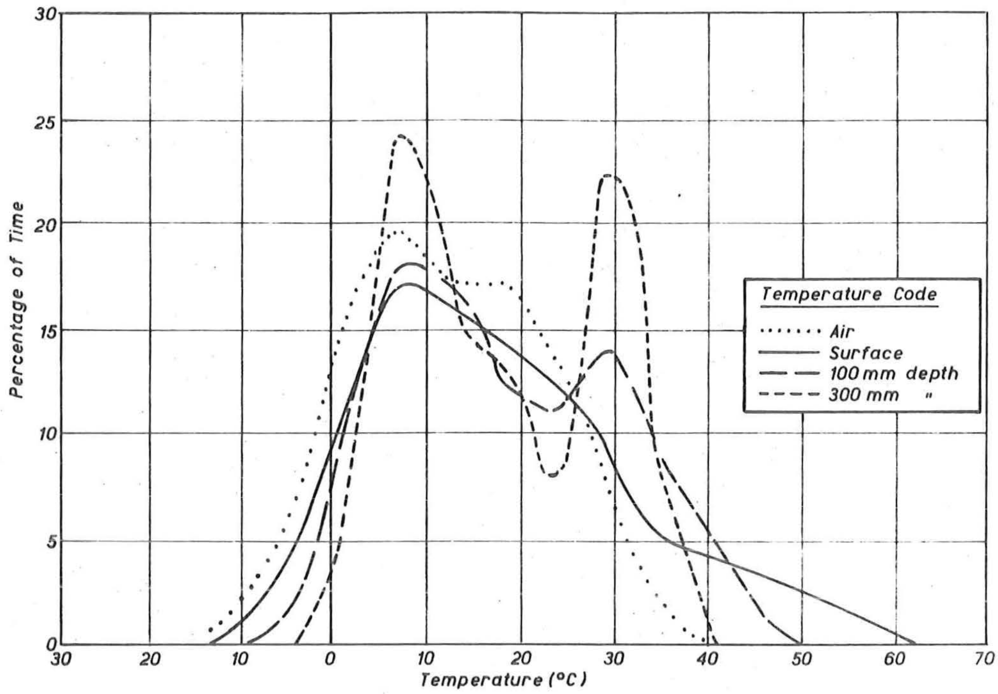


Fig. 3.7 DURATION OF TEMPERATURE LEVELS AT VARIOUS DEPTHS IN A 300 mm ASPHALT CONCRETE PAVEMENT IN MARYLAND, U.S.A. (after Kallas[85])

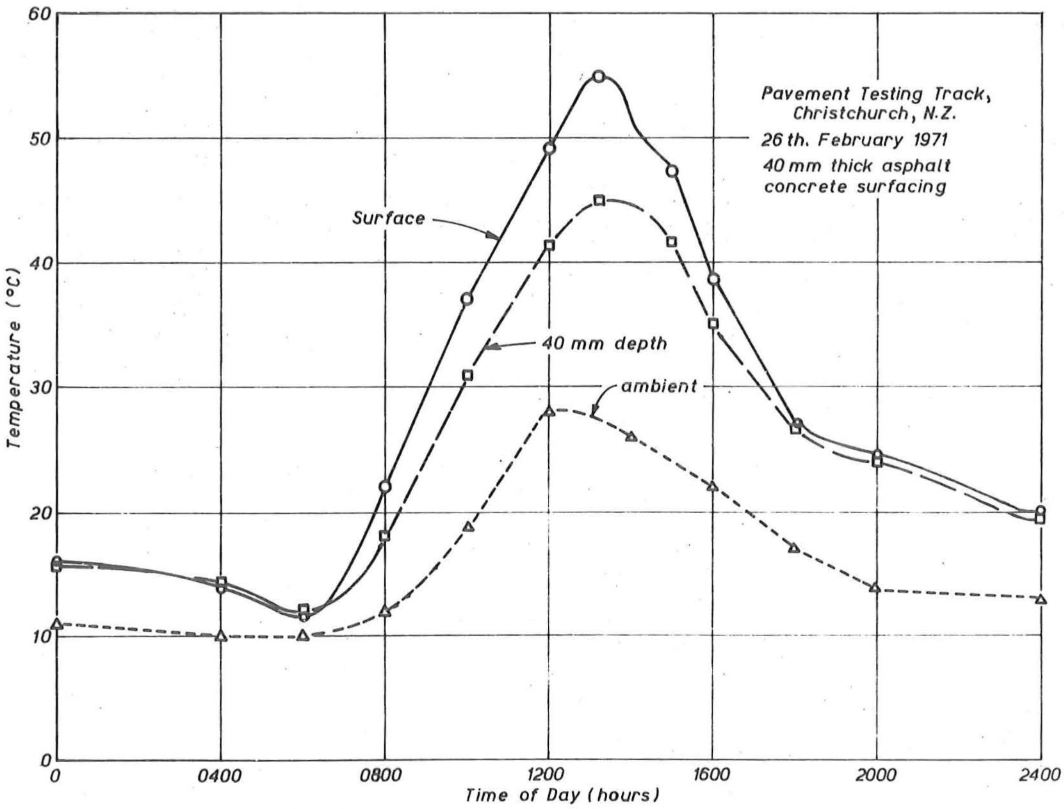


Fig. 3.8 TEMPERATURE RECORD FOR ONE SUMMER DAY IN CHRISTCHURCH, NEW ZEALAND.

3.4.2 (cont.)

hottest day for which a record was obtained are shown in Figure 3.8. The maximum ambient temperature that day was 28°C which is 6 C° below the maximum summer temperature. Hence the maximum surface temperature is probably approximately 64°C in sheltered reflective conditions or 60°C in normal conditions. The incidence of high pavement temperatures is likely to be greater in the north of New Zealand due to warmer average temperatures but it is unlikely that maximum surface temperatures will be any higher.

There are numerous other reports on pavement temperatures but the foregoing results give a reasonable picture. The crux of the problem now lies largely in the realm of the relationship of peak traffic loading to peak temperature conditions. This relationship is exceedingly difficult to assess since it is so dependent on season, geographical location, work-time patterns of various industries, etc. However a general pattern might be such as in Figure 3.9 with the peaks of traffic and temperature occurring during approximately the same period, i.e. morning to mid afternoon. Hence the severest conditions of loading and material properties are likely to overlap.

3.4.3 Oxidation

A film of bitumen hardens over a period of time due to two major factors. There is a chemical oxidation process caused by exposure to the atmosphere and there is also a work-hardening effect caused by repeated stress-relaxation cycles, the latter having been mentioned in Chapter Two.

The oxidation rate of the surface course binder appears to be largely dependent on the air voids existing in the asphalt concrete when constructed[91]. Goode and Owings[92] traced the reduction of binder penetration over a period of four years for six pavements with a wide range of initial air void contents. Their results are reproduced in Figure 3.10 with a high degree of hardening shown as a small retained percentage of penetration. The asphalt used in each case was originally 85/100 penetration grade, although that used for section 6, the stray result, showed high susceptibility to hardening in a thin-film oven test. The general trend is for the binders in pavements with high initial air void content to harden rapidly. If this curve is extrapolated to estimate the condition where no hardening is likely to occur this turns out to be approximately 3% air voids[2].

3.4.3 (cont.)

Ductility is also greatly reduced in time for pavements of high initial void content.

In any event, oxidation will be a long-term process especially since most asphalt concrete pavements today are in either the medium or low initial air voids category. Hence any accelerated testing method must either avoid this problem or treat it artificially.

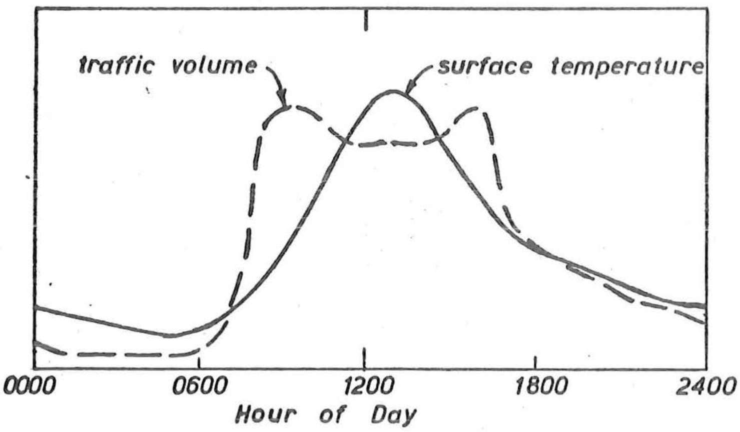


Figure 3.9 RELATIONSHIP OF TEMPERATURE AND TRAFFIC DISTRIBUTIONS

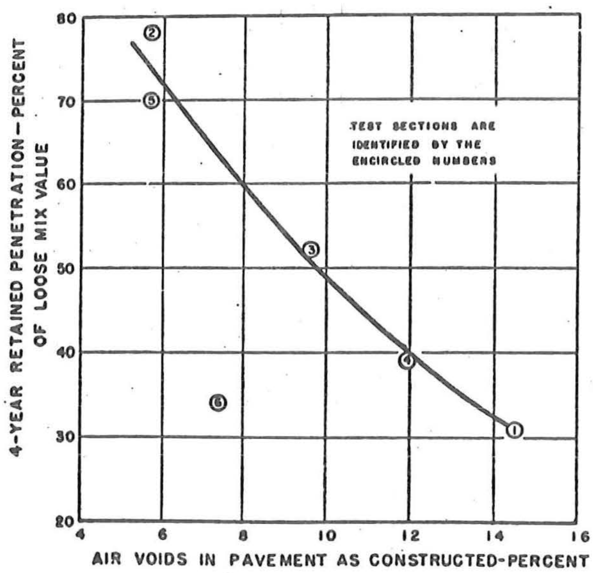


Figure 3.10 EFFECT OF AIR VOID CONTENT ON BINDER OXIDATION (from Goode and Owings[92])

CHAPTER FOUR

CHOICE OF TESTING MODE
- THE PAVEMENT TESTING TRACK

This chapter describes the factors leading to the choice of a large scale testing track as the simulation mode to be used for investigating the problem of traffic compaction. A description is given of the main features of the track.

4.1 CHOICE OF MODE OF SIMULATION

4.1.1 Summary of the Problem

The first three chapters have made the scope and complexity of the problemclearer. The action of vehicular traffic on a pavement modifies some of its material properties and consequently alters its structural properties. In particular, an asphalt concrete surface course appears to densify at a diminishing rate after construction due to the passage of traffic over a period of several years. The resistance to deformation of asphalt concrete due to imposed stresses is greatly reduced by any of several factors, especially a high content of air voids, a soft binder, or a rounded aggregate. By causing a reduction in air voids, i.e. compaction, the action of traffic is increasing the resistance to deformation of the asphalt concrete to a stage that some persons term "ultimate density", a density at which the resistance to deformation is equal to, or greater than, the imposed stresses. Because asphalt is a viscous substance it is temperature sensitive and high pavement temperatures reduce stability. Crushed aggregates, being of angular shape, can develop a strong interlock and hence the common use of crushed aggregates in New Zealand may produce mixes more resistant to compaction by traffic. The imposed loading takes the form of a parabolic pressure distribution over an elliptical or nearly rectangular area. The imposed normal stresses are modified only slightly over the thickness of the surface course by the flexure of the underlying structure. However vertical deflections and horizontal stresses and strains are higher for a flexible base than for a rigid base.

The problem therefore becomes one of determining the behaviour of a selected range of mixes with the following considerations:-

(i) mixtures must be realistic, preferably full scale

(ii) construction conditions affect material properties

(iii) thermal environment - controlled

(iv) imposed loading - realistic, controlled and preferably full scale

(v) uniform known base conditions.

What will be the most satisfactory testing mode in this case?

4.1.2 Existing Test Methods

(a) Full-scale tests and field trials

The closest that one can approach all conditions for a pavement research problem is by constructing a full scale pavement and subjecting it to normal traffic. However, the former is extremely expensive especially for testing more than one set of parameters, and the latter is extremely difficult to control. Furthermore it involves long periods of several years. Examples include the A.A.S.H.O. Road Test[93], the Tunnel Road, Christchurch, New Zealand[94], the Shell Avenue Test Road[95] and many others.

For the present problem, the laying of several trial strips on trafficked highways would be neither too expensive nor too inconvenient. However the imposed conditions could not be controlled and the factors of imposed loading and temperature could not be isolated and assessed: an unsatisfactory situation for this research.

(b) Small scale tests

Many attempts have been made to design accelerated testing facilities, some of them on a small laboratory scale and some of them very elaborate. Gyratory machines provide a controlled kneading action and produce high densities similar to those achieved under heavy traffic[96,97]. There are many triaxial type cells including the stabilometer[98] and some more sophisticated devices with controlled cyclic normal and shear stresses[99]. Reciprocating linear machines consist of a tyred wheel travelling along a linear path usually with controlled speed, load and lateral location and with a lift and return mechanism[100,101]. Continuous tracking devices take the form of miniature circular test tracks with a tethered wheel travelling around the track[102,103,104,105].

All those laboratory tests however are unsuitable in various aspects. In particular, the problem of scaled-down dimensions is always present and the loading conditions and stress-fields are being artificially simulated one way or another. The tests do, of course, furnish particularly valuable information on the material properties of bituminous materials but none of the tests were considered to be well suited to this present research.

4.1.2 (cont.)

(c) Large scale tests

Several large scale outdoor test tracks have been built to overcome the dimensional problems encountered at laboratory scale[106,107]. Such large scale testing can combine good "laboratory control" of materials, loading and, to some extent, environmental conditions, with the full scale dimensions of a trafficked highway.

4.1.3 Choice of a Testing Track

After consideration of all the available modes of testing, the mode that best suited the present research was an outdoor testing track of modestly large scale. The reasons for this decision were:

(i) Normal field paving and rolling procedures could be used.

(ii) Normal surface course dimensions could be used: mixture, thickness, width, etc.

(iii) Full scale wheels could be used for a realistic imposed loading.

(iv) Repetitive loading and vehicle speed could be realistically controlled.

(v) Uniform base conditions would be comparatively easy to attain.

(vi) Several different mixtures could be tested at once and in a short period of time.

(vii) Environmental conditions: temperature could be controlled for short periods and the complication of oxidation could be avoided by testing over a short period.

(viii) Protection and convenience for instrumentation.

(ix) Such a facility is a versatile and continuing asset to road research in New Zealand.

As a result of this decision early in 1968, the University of Canterbury Pavement Testing Track (see Frontispiece) was built and became operational in April, 1969.

4.2 THE PAVEMENT TESTING TRACK

Only an outline of the general features of the Pavement Testing Track and relevant comments on its performance are described in this section. The design, construction and performance of the Testing Track are described in detail in Appendices I and II, and have been described briefly elsewhere[108,109].

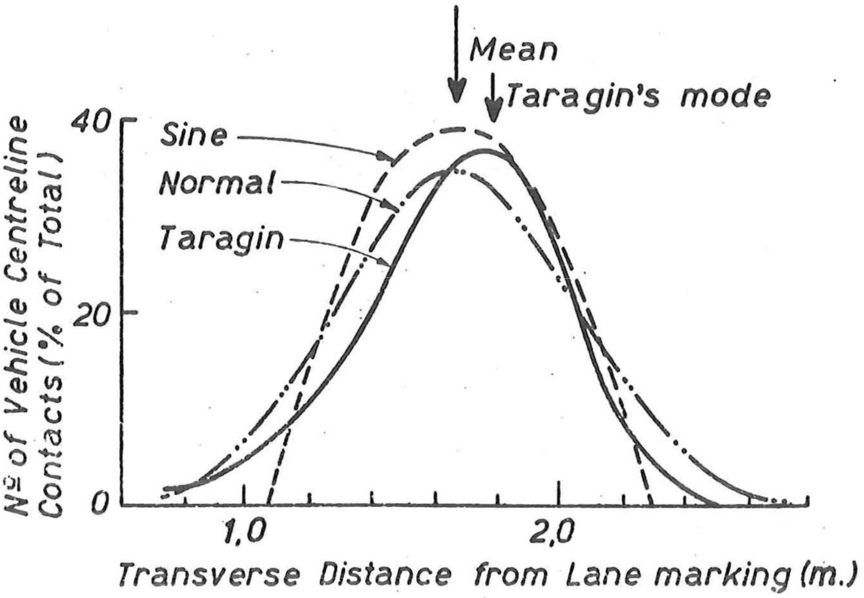
4.2.1 General Features

The Pavement Testing Track is sited on the outskirts of Christchurch and approximately 8 kilometres from the University.

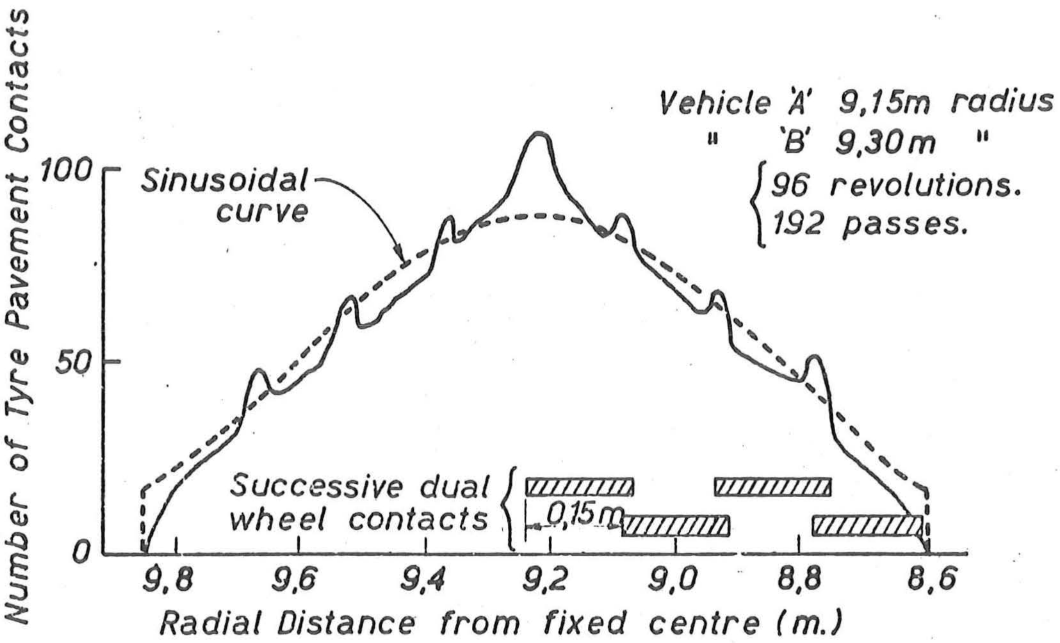
The Testing Track has a mean diameter of 9.1 m (30 ft) and is trafficked by a pair of diametrically opposed dual-wheel Testing Vehicles attached to a central pivot, see Frontispiece. Each Vehicle is capable of carrying a maximum load of 40 kN which is equivalent to a 80 kN single axle load, the legal maximum in New Zealand and the internationally accepted Design Axle Load. The load on each Vehicle is variable and is applied by ten concrete ballast blocks each weighing 2.7 kN. The Vehicles are driven at a fixed speed of 18.7 km/h by an electric brake motor attached to one Vehicle and driving through one wheel. The other wheels rotate freely and independently in order to minimise slip forces. A traversing motion superimposed on the circular motion of the Vehicles by the action of an eccentric centre creates a lateral distribution of trafficking across the path similar to that existing on normal highways; cf. section 3.2.2 and see Figure 4.1. The traversing pattern repeats itself after 96 revolutions and the width of the consequent trafficked path is 1.2 m.

The bed of the Testing Track comprises eight straightsided portland cement concrete slabs forming an octagonal annulus. Each slab is 2.59 m (8 ft 6 in) wide, has an outside length of 8.3 m, averages 280 mm in depth, and incorporates a crossfall of 1:50.

Heating elements embedded in the concrete slab at a depth of 50 mm provide a measure of control over the temperature of the surface course test material. The elements cover only the immediate area of the trafficked path and provide a controlled incremental heat input up to a maximum of 936 w/m^2. A 250 mm thick layer of no-fines coarse aggregate beneath each concrete slab acts as an insulator.



(a) Comparison of Highway and Simulated Distributions



(b) Distribution for Testing Vehicle

Figure 4.1 LATERAL DISTRIBUTION OF TRAFFICKING FOR TESTING VEHICLE

4.2.1 (cont.)

The Testing Track is enclosed by a fence 2 m high which acts as both a protective barrier and a windshield to reduce convection heat losses. The Vehicle and the heating elements are controlled from a switchboard located in the Control Hut outside the Track.

4.2.2 Comments on Performance

(a) Rigid Base

A concrete base is more rigid than normal highway base construction and this introduces a departure from common field conditions. However the elastic analysis employed in section 3.3 showed that vertical stresses at low to mild temperatures are only of the order of $1\frac{1}{2}\%$ higher on a rigid base than on a flexible one. At high pavement temperatures the difference drops to the order of 0.2%. Horizontal stresses are more affected by base flexibility than vertical stresses but they are generally of second order magnitude compared with normal stresses.

There were three main reasons for the decision to construct a "rigid" concrete base for the test bed rather than a "flexible" granular base:

(i) Only a permanent rigid base allows the safe installation and performance of electric heating cables.

(ii) A concrete base of sufficient thickness provides a base support which is uniform around the track and which will remain so during its useful life.

(iii) Materials laid on a concrete base are easily removed after testing without the disturbance and need for relaying of the base.

(b) Temperature Control

Natural solar heating was used as much as possible in controlling pavement temperatures since use of the heating elements tended to reverse the temperature gradient in the surface course. The elements were used principally as a fine control to attain and maintain the desired testing temperature. They were also used to preheat the pavement and thus extend the duration of testing at the

4.2.2 (cont.)

desired temperature level.

Heat input was satisfactorily uniform when the two elements in each slab were used together but some non-uniformity was encountered when only one element was being used. In two of the slabs, No. 2 and No. 6, the elements short-circuited after one and two years respectively causing local overheating. Thereafter the heating on these slabs was used rarely and cautiously.

(c) Vehicle Speed

The speed of the Testing Vehicles, 18.7 km/h, is considerably slower than the speed of normal highway traffic, 40 to 100 km/h. This means that the duration of loading on the track is between 2 and 5 times longer than for normal traffic conditions. This longer loading duration affects the behaviour of bituminous materials and is likely to cause an effective acceleration of trafficking.

The trafficking frequency of the Testing Vehicle is approximately 11 axles per minute but the loading frequency will be less than this taking the traversing motion into account. On a normal highway the time delay between successive wheel loads ranges from 0.06 seconds for a tandem axle vehicle at high speed to say 30 seconds between successive vehicles on a moderately trafficked road. These examples are equivalent to a range of trafficking frequency on normal highways of between 900 and 2 axles/minute but the average frequency is probably of the order of 10 to 20 axles/minute. Hence the Testing Vehicle frequency of 11 axles/minute is as satisfactory a simulation as could be expected.

(d) Tyre Slip

The eccentric circular motion of the Testing Vehicle causes slip over the tyre contact area because the direction of motion is changing continuously. The tyre slip varies from a minimum of 0.8% to a maximum of 2.2%. The maximum commercially recommended with respect to tyre wear was 0.2% and hence the slip for the Testing Vehicle must be classified as severe and horizontal surface stresses are expected to be high. The situation is not as severe as cornering for normal vehicles, however, because the whole of the

4.2.2 (cont.)

centripetal force necessary to change the direction of motion of the Vehicle is provided by the diametral arm. Hence the commercially recommended maximum is considered too low for this situation and the surface stresses will be less severe than implied because the centripetal force is not being fully transmitted through the tyre walls. The slip could have been reduced by increasing the radius of the Testing Vehicle but this was prevented by financial restraints.

(e) General

The experimental results obtained from the Pavement Testing Track will be the ultimate test of the facility as a mode of simulation. However on a simple performance basis the facility has been highly satisfactory and performed its function adequately.

CHAPTER FIVE

OUTLINE OF EXPERIMENTAL INVESTIGATIONS

This chapter describes the investigations conducted on the pavement testing track and two sections of normally trafficked highway. The selection of parameters and the testing procedure are outlined.

5.1 GENERAL APPROACH TO INVESTIGATIONS

5.1.1 The Pavement Testing Track

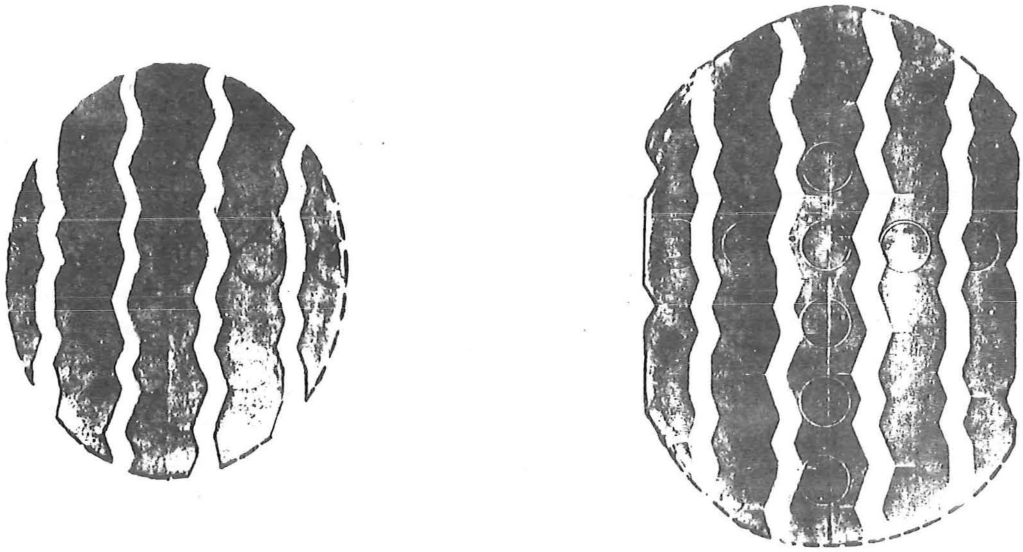
The reasons advanced in Chapter Four leading to the selection of a large scale pavement testing track for the investigations were centred on the desire to control as many conditions as possible. This, combined with the important factors emerging from the earlier chapters on the behaviour of asphalt concrete as a material and the physical context of the surface course in the pavement system, largely defined the approach to the investigations.

(a) Loading

The loading factor which can neither be controlled nor easily measured on a highway was easily controlled on the Testing Track. The normal contact pressure was used as the basic measure and the range of common vehicle contact pressures was simulated as well as possible by choosing the two extremes available with the Testing Vehicle. The distribution of contact pressure was made as uniform as possible, i.e. a slightly under-inflated condition, so that the loading magnitude could be easily analysed.

Contact pressures were measured by pressure cells developed especially for this task. The design, characteristics and use of the cells are described in Appendix III.

The resulting pressure conditions which were adopted for the minimum and maximum vehicle loads are shown in Figures 5.1 and 5.2. For the maximum 20 kN wheel load, equivalent to a 80 kN dual wheel single axle load, an inflation pressure of 520 kN/m^2 produced an almost uniform contact pressure with 620 kN/m^2 peak under the tread with an average of 448 kN/m^2 over the total enclosed contact area of 44200 mm^2. For the minimum 6.5 kN wheel load, equivalent to a 26 kN dual wheel single axle load, an inflation pressure of 240 kN/m^2 gave a more parabolic transverse distribution with a peak of 320 kN/m^2 under the centre tread and an average of 247 kN/m^2 over the total enclosed contact area



(a) Light Load (b) Heavy Load

	AREA (mm^2)	CONTACT PRESSURE (kN/m^2)	LOAD N
A	1490	100	150
B	5760	280	1620
C	7300	320	2340
D	5860	280	1650
E	2140	120	260
TOTALS	22550	-	6020

	AREA (mm^2)	CONTACT PRESSURE (kN/m^2)	LOAD N
A	4460	540	2410
B	7360	600	4420
C	8810	620	5470
D	7390	620	4590
E	4340	640	2780
TOTALS	32360	-	19670

		(a)	(b)
Total enclosed area	:	31200	44200 mm^2
$\frac{\text{Treaded area}}{\text{Enclosed area}}$	:	71	73 %
Total actual load	:	6600	20000 N
Average contact pressure over encl. area	:	210	450 kN/m^2
Radius of equivalent circular area	:	90	118 mm
Inflation Pressure	:	240	520 kN/m^2
Representative Contact Pressure	:	280	620 kN/m^2

FIGURE 5.1 TYRE CONTACT AREAS FOR TESTING VEHICLE

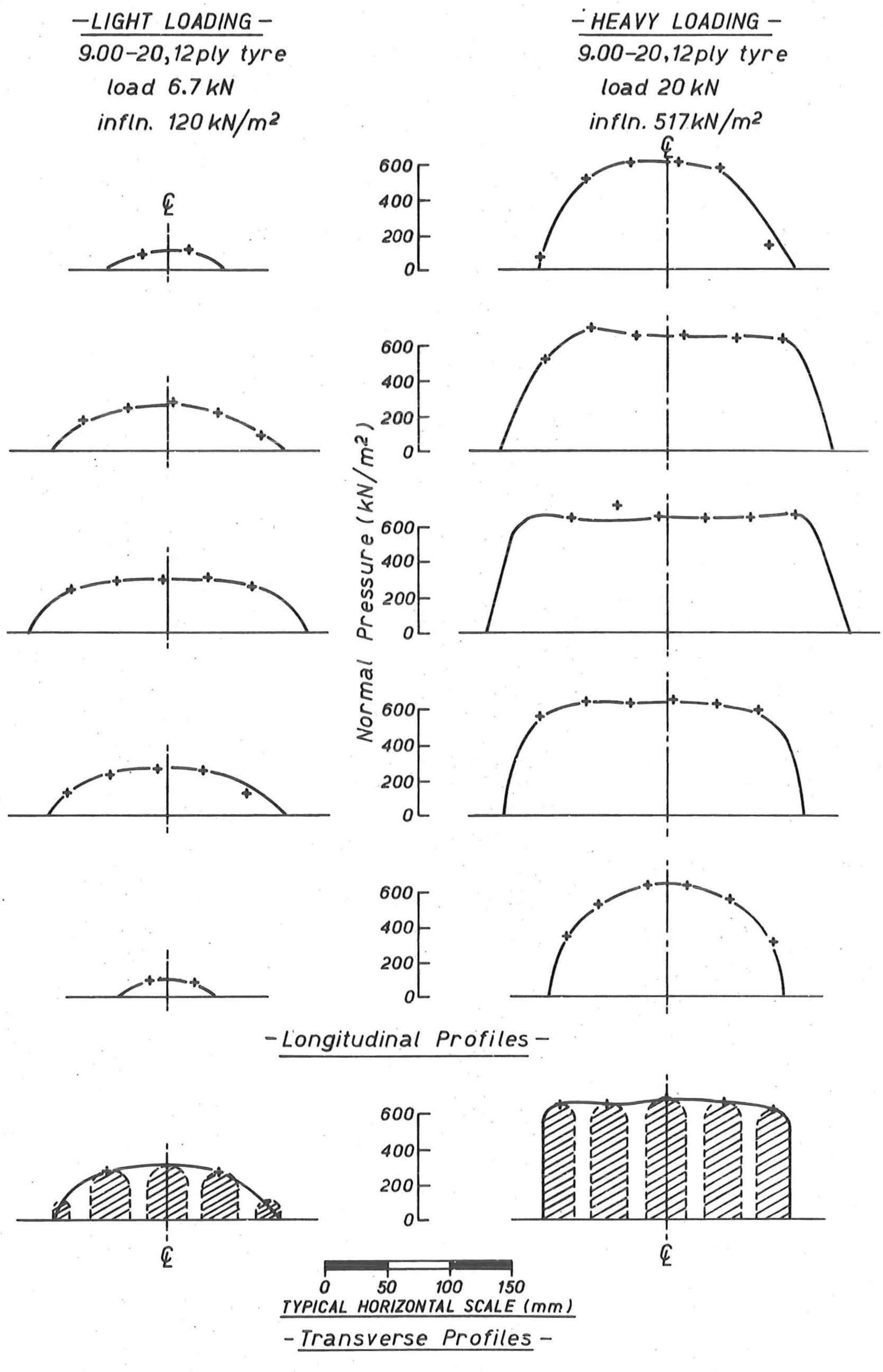


Fig. 5.2 CONTACT PRESSURES FOR TESTING VEHICLE

5.1.1 (cont.)

of 26300 mm^2. The representative contact pressure for the minimum load was taken to be 280 kN/m^2. Note the much more oval and nearly circular shape of the contact area for the lightly loaded large tyre compared with the more rectangular shape for the fully laden tyre. Light passenger cars may well have contact pressures as low as 180 kN/m^2, but 280 kN/m^2 was considered to be a satisfactory representation for light vehicles. Heavy vehicles may produce pressures up to 700 kN/m^2 but their incidence is generally very low. The one characteristic of the tyre-contact that could not then be changed was the size and shape of the contact area but this was considered to be of secondary importance to the magnitude of the contact pressure.

The measured results compare well with the findings cited in section 3.2.2. The contact pressure at the centre of the contact area is about 30% higher than the inflation pressure. Due to the aforementioned limitations of the cells for measuring low contact pressures no specific measurements were taken for small vehicles. However, for common inflation pressures of the order 125-165 kN/m^2, the average contact pressures are likely to be of the order of 150-200 kN/m^2.

The dynamic situation will not be very different. The normal pressure at the leading edge of the tyre contact is probably about 2% higher and the contact area very slightly longer, but neither of these will significantly affect the subsequent findings. The speed of the testing vehicle is fixed and, although 19 km/h is slower than most traffic speeds, this constraint had to be accepted. The findings mentioned in section 3.3.3 will help to assess the effect of this on the results.

(b) Temperature

The satisfactory simulation and control of temperature is a difficult problem. Even in a long-term full-scale highway study

5.1.1 (cont.)

the timing of the seasonal variations may have significant effect. Because the Testing Track afforded some control of temperature it was decided to limit testing to only two temperature levels, one representing cool conditions and the other hot summer conditions.

On the basis of the information in section 3.4.2 and some observations during preliminary tests at the track, the author chose temperature levels of 25°C and 40°C measured at the middepth of the surface course. The lower temperature, 25°C, was intended to cover the maximum pavement temperature likely to occur during 94% of the year in New Zealand. Temperatures between 25°C and 40°C may be expected to occur for about 5% of the year (i.e. about 430 hours) and temperatures exceeding 40°C occur during probably about 1% of the year (i.e. about 90 hours).

Since asphalt concrete has least resistance to compaction at high temperatures, the test temperatures have also been chosen on the high side. Performance above 40°C may be critical at some stages of the life of the pavement but such temperatures occur so rarely in New Zealand that separate consideration should be made of them.

(c) General Suppositions

The selection of two conditions each of loading and temperature with the constant condition of "rigid" base support provides four combinations of physical testing conditions. No more combinations could be employed without unduly extending the length of a test series beyond four months.

The order of severity of these four combinations was considered to be the following:-

(i) low load, low temperature,
(ii) low load, high temperature,
(iii) high load, low temperature,
(iv) high load, high temperature,

although there was some uncertainty regarding the order of (ii) and

5.1.1 (cont.)

(iii). The policy of gradually increasing the severity of loading conditions will result in the maximum bearing strength being attained, cf. section 2.3.4(a). If the pavement were first subjected to the severest condition, (iv), it would behave differently and, if it were to "fail", it would "fail" prematurely. The preliminary tests conducted on the track showed this spectacularly when a strip at a temperature of $60^{\circ}C$ subjected to heavy loads deformed very badly after only 700 vehicle passes, q.v. section 5.3.6. Even after such extreme deformation the same strip proved to be stable under less severe conditions. Similar material when tested in the aforementioned standard sequence did not exhibit such spectacular deformation.

Behind the choice of all these conditions there lies a general philosophy. It is expected that under any one set of these conditions the surface course will compact under trafficking until it reaches a "stable state" for that set of conditions. Such a "stable state" is defined as those material conditions which are developed through the process of trafficking under certain constant testing conditions and which do not then alter under further trafficking. The author uses this terminology in preference to the term "ultimate state" since the state is largely dependent on the general loading conditions and cannot be "ultimate" in the correct sense of the word while the loading conditions could increase in severity. On this supposition it should then be possible to compare the effects of load and temperature by the properties of the asphalt concrete at these stable states. Furthermore, by conducting all stages of a test series within three months, oxidation effects should be insignificant.

That then is the formulation of the physical and loading conditions which were maintained throughout the testing programme. The material properties of the test mixes then remained to be chosen. Since each test series was expected to last the three months of a summer season, only two series could be planned in this project extending it to a duration of four years.

5.1.2 Highway Studies

The necessarily artificial conditions of the testing track provide excellent control but the question arises, 'How realistic is this?'. Comparative test areas on two highway sites near Christchurch were studied to answer this query, especially regarding the flexibility of the base. One of these was laid on the Main North Road in Christchurch at the beginning of this project in February, 1968. The original purpose of this test area was to study the influence of two mix design parameters but later it also provided an answer to the question above. The other test area was laid on the Christchurch Northern Motorway in December, 1970 with the special intention of studying the effect of base flexibility. These highway test areas are discussed in detail in section 5.5.

5.2 PARAMETER SELECTION FOR TRACK TESTS

5.2.1 Important Material Parameters

The eight slabs of the pavement testing track permit the simultaneous testing of eight combinations of material parameters. With two test series planned this gave a total of sixteen possible combinations. The parameters to be considered fall into two categories: those in the mix design and those in construction; both affecting the behaviour of the material.

The most important factors in mix design which emerge from section 2.1 appear to be binder content and viscosity, and aggregate structure with the associated property of air voids. Aggregate gradation per se seems of lesser importance beside these others once the general type of gradation has been decided. Although gradation has often been a controversial issue the author decided to choose one well-proven acceptable gradation, within the current specifications, for all the tests. This meant a dense gradation, as commonly used in New Zealand. This then permitted more variations of the other parameters.

Paving and rolling procedures were to be as similar as possible to those normally employed. Here, the important parameter was the minimum density which should be avhieved during the construction phase: does it affect the final density achieved after trafficking? The surface course thickness was another construction factor of interest

5.2.1 (cont.)

because of its possible influence on the effective stability of the asphalt concrete. If thickness was to be a parameter then the maximum aggregate size had also to be considered because the relative dimensions of these seemed likely to have some interdependence with regard to stability.

The following parameters were therefore selected for the test series (with the number of variations in parentheses): binder content (3), binder viscosity (2), construction density (2), surface course thickness (4) and maximum aggregate size (2). The combinations of these which were tested are described below.

5.2.2 Test Series 1

In the first series, the binder content and viscosity and the construction density were varied. The aggregate gradation was kept constant and was designed to replicate that laid on the Main North Road test strip, Figure 5.3.

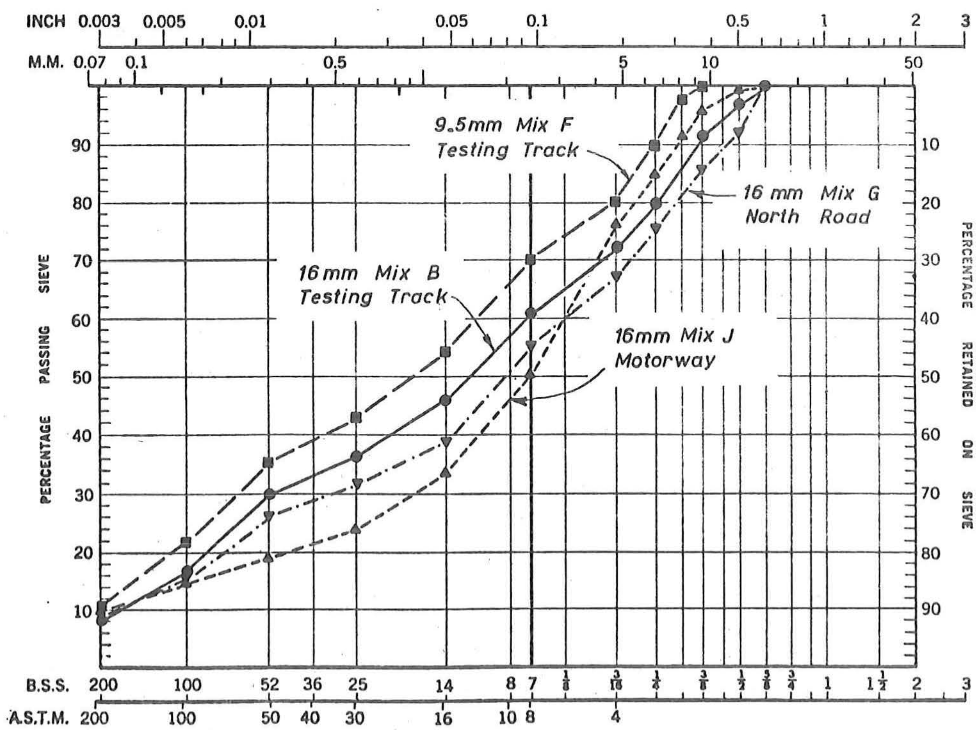


Figure 5.3 AGGREGATE GRADATIONS FOR ALL TESTS

5.2.2 (cont.)

Two binder viscosities were chosen to demonstrate the influence of viscosity on traffic compaction. From this a measure of the effects of oxidation may also be obtained. One was an 80/100 penetration asphalt which is the softest grade normally used in asphalt concretes in New Zealand and the other had a penetration of 180/200. Their viscosity-temperature properties are included in Appendix V.

The mixes were designed by the Marshall method and three binder contents were chosen for each asphalt to give a range of above-optimum, optimum, and below-optimum, q.v. Appendix V for details. The properties of the five mix designs (A-E) used are shown in Table 5-I.

The variation of construction density was limited to two levels by the number of combinations able to be tested in the series. These were chosen at 98% and 95% of Marshall 75-blow density thus representing the common range expected in practice. Densities below 95% have been well proved as unsatisfactory overseas and densities above 98% are sometimes difficult to obtain economically during construction.

The combinations of these three variables used in the Test Series 1 are shown in Table 5-II.

5.2.3 Test Series 2

The final selection of parameters for the second test series was not made until the results of the first series were known. However, the main aim was to test the significance of the dimensional effects of maximum aggregate size and surface course thickness.

Two maximum aggregate sizes were chosen, 16 and 9.5 mm. The gradation of the 9.5 mm aggregate was very similar to that of the 16 mm aggregate which was the standard gradation used in Series 1 and on the parallel highway tests, Figure 5.3.

The layer thicknesses were chosen to be approximately two, four and six times the maximum aggregate size. Hence thicknesses of 19, 38 and 64 mm for the 9.5 mm mix, and of 38, 64 and 100 mm for the 16 mm mix were chosen. Additional strips of the 16 mm mix were chosen at thicknesses of 38 and 64 mm to be laid at 95% laboratory density in

TABLE 5-I

MIX CHARACTERISTICS FOR ALL TESTS

Code	Asphalt pen. grade (mm)	Asphalt Content (% wt)	Max. Agg. Size (mm)	Air Voids (%)	Agg. Voids V.M.A. (%)	Voids Filled VFA (%)	Marshall Stab. kN	Marshall Flow 0.1mm	Z.A.V.* Density kg/m^3
	PAVEMENT TESTING TRACK - DESIGN								
A	80/100	6.8	16	2.5	18.0	86.4	9.8	29	2396
B	80/100	6.1	16	4.0	17.8	77.5	10.7	26	2420
C	80/100	5.2	16	6.1	17.7	65.5	12.9	23	2452
D	180/200	6.1	16	3.5	17.4	80.0	8.9	23	2420
E	180/200	5.2	16	5.6	17.3	67.7	8.9	20	2452
F	80/100	6.75	9.5	3.7	19.0	80.5	10.8	26	2403
	MAIN NORTH ROAD TEST AREA - CONSTRUCTED**								
G	80/100	6.2	16	3.85	18.3	79	15.0	28	2415
H	180/200	6.15	16	4.3	18.3	77	9.7	23	2418
	CHRISTCHURCH NORTHERN MOTORWAY - CONSTRUCTED**								
J	80/100	6.1	16	2.1	16.8	87.5	14.9	30	2420

*Z.A.V. = Zero Air Voids **Average values

TABLE 5-II

DESIGN PARAMETERS FOR ALL SERIES OF TRACK TESTS

Track Strip *	Mix Code	Binder Viscosity	Binder Content	Max.Agg. Size (mm)	S.C. Thickness (mm)	Nom.Constr. Density (% lab.)
TEST SERIES 0						
02	D	Low	Optm	16	50	–
04	D	"	"	"	"	–
01	D	"	"	"	"	98
07	D	"	"	"	"	95
TEST SERIES 1						
11	B	High	Optm	16	50	98
12	A	High	Rich	"	"	98
13	B	High	Optm	"	"	95
14	A	High	Rich	"	"	95
15	C	High	Lean	"	"	98
16	D	Low	Optm	"	"	98
17	C	High	Lean	"	"	95
18	E	Low	Lean	"	"	98
TEST SERIES 2						
21	B	High	Optm	16	38	98
22	B	"	"	16	64	98
23	B	"	"	16	100	95
24	F	"	"	9.5	64	95
25	B	"	"	16	64	92
26	F	"	"	9.5	38	95
27	B	"	"	16	38	92
28	F	"	"	9.5	19	95

*The first digit is the number of Test Series, the second is the Track Slab number.

5.2.3 (cont.)

order to check an apparent dependence on initial construction density of this mix in the first test series.

The mix design and construction parameters for Series 2 are summarised in Table 5-II.

5.3 PRELIMINARY TRACK TESTS

5.3.1 Introduction

During May and October of 1969 some preliminary tests were conducted at the Testing Track. The purposes of these tests included:- (1) to give the testing facilities a trial run; (2) to make running adjustments to the vehicle, q.v. Appendix I; (3) to develop temperature measurements and determine the heating response; (4) to attempt to reproduce the mix laid on the Main North Road Test Area; (5) to test paving procedures on the Testing Track for small tonnages; (6) to examine construction rolling on the Track and how to control the constructed density; and (7) to determine the relative load, temperature, traffic, density and time scales for track testing.

The test materials were laid on slab numbers 2 and 4 for the May tests and numbers 1 and 7 for the October tests. By a standard nomenclature that was adopted these test strips were numbered 02, 04, 01 and 07 respectively. The first digit refers to the number of the Test Series: 0 for the Preliminary Tests, and 1 and 2 for the first and second series. The second digit refers to the number of the slab on the Track indexed in Appendix Figure I.1.

The conclusions from the preliminary tests are summarised below.

5.3.2 Temperature and Heating Response

Temperatures were measured by iron/constantan thermocouples of 18 and 20 gauge connected to a 24-channel strip chart recording potentiometer. Several other means of measurement were considered including a mercury-in-glass thermometer in an oil-filled hole, a needle pyrometer, and a needle thermistor but these would have required interruption of testing and manual point measurement. The technique developed for embedding the thermocouples is described in Appendix V.

5.3.2 (cont.)

The time elapsed before a temperature response to heating input occurred at the surface of the concrete base slab was approximately two hours. With maximum heating switched on, the time elapsed before the maximum pavement temperature was reached was about 12-15 hours depending on ambient conditions. When the heating input was reduced, about one hour elapsed before the temperature responded although it was about 9-10 hours before equilibrium was reached.

5.3.3 Construction

Normal field procedures were used for mix production, paving and compacting and details of these are included in Appendix V.

The control over pavement thickness and density that was achieved is demonstrated in Figures 5.4 and 5.5. Thickness control was not very good with a variation of 62-38 mm for a specified 50 mm thickness, Figure 5.4. Although much of this could be due to variations in the finished level of the concrete base, greater care needed to be observed on the manual thickness control.

The density distribution, Figure 5.5, also shows a large variation amongst samples from either strip. The variation is greater transversely than it is longitudinally. This arose from the difficulty in obtaining an even coverage by a pneumatic roller over such a narrow strip. In particular, the half metre strip near each edge could not be adequately compacted without some lateral support being given to the mix , e.g. Strip 02.

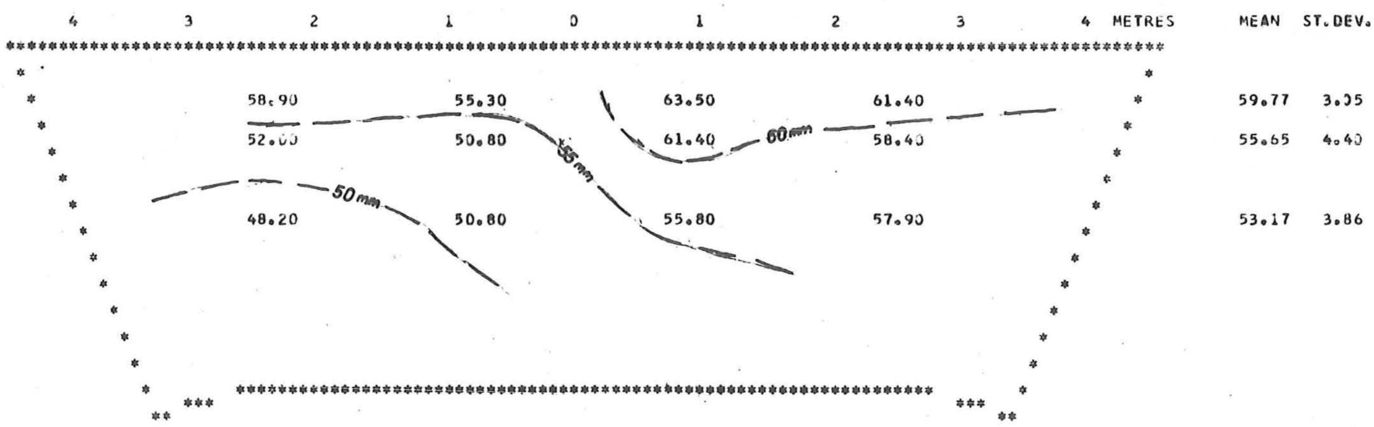
5.3.4 Trial Trafficking

In the May tests, core samples were cut from strips 02 and 04 after 270 passes of the 40 kN vehicles at varied ambient temperatures. This was the period of running-in and adjustments to the vehicles. Even after this small amount of traffic at relatively cool to mild temperatures there was a significant increase in density of 0.5 and 0.9% respectively, Table 5-III.

The second period of trafficking was conducted at the very high middepth temperature of $60^{o}C$. After 730 passes of the 40 kN vehicles (i.e. approximately one hour) air voids had decreased to 1.4% and 1.6% respectively and severe rutting was noted in both

TEST TRACK STRIP 02 PLAN OF THICKNESS AFTER CONSTRUCTION

MIX DESIGN CODE D SPECIFIED THICKNESS 50 MM.

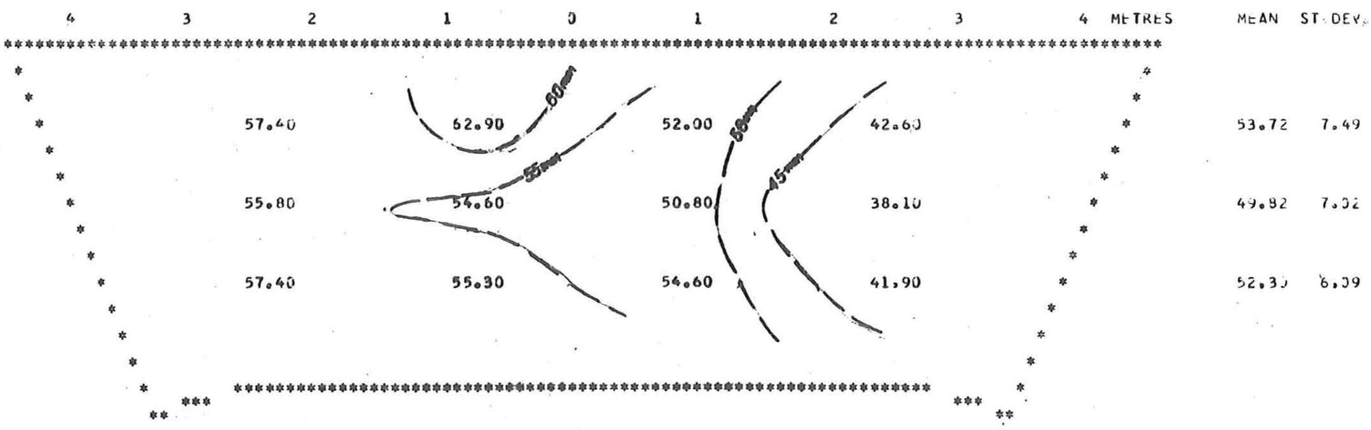


MEANS	53.03	52.30	60.23	59.23
STD.DEVNS.	4.43	2.12	3.25	1.55

MEAN OF ALL VALUES 56.20 mm
STANDARD DEVIATION 4.68
NUMBER OF SAMPLES 12.

TEST TRACK STRIP 04 PLAN OF THICKNESS AFTER CONSTRUCTION

MIX DESIGN CODE D SPECIFIED THICKNESS 50 MM.



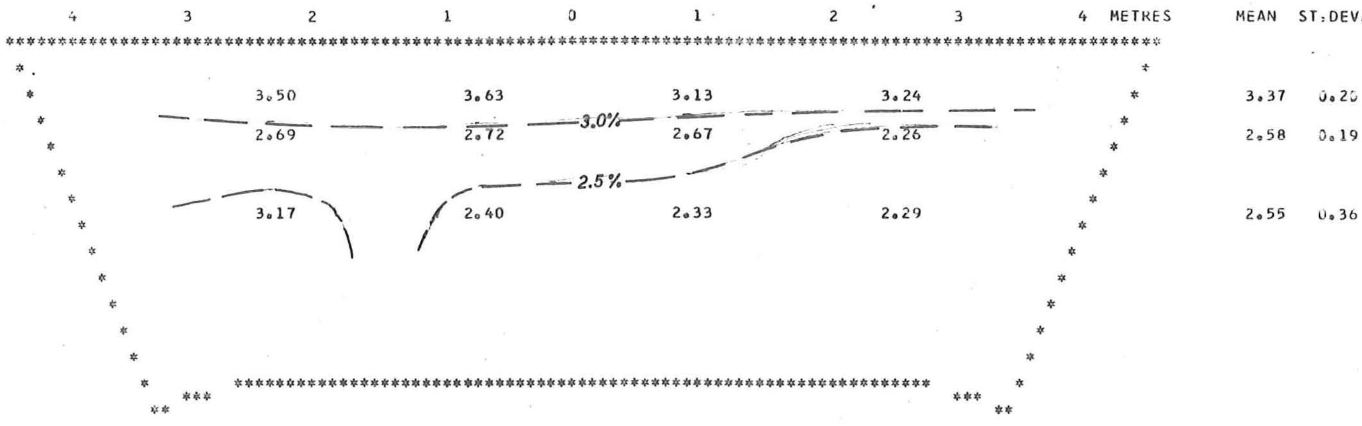
MEANS	56.87	57.60	52.47	40.87
STD.DEVNS.	0.75	3.76	1.59	1.98

MEAN OF ALL VALUES 51.95 mm
STANDARD DEVIATION 7.08
NUMBER OF SAMPLES 12.

Figure 5.4 VARIATION IN CONSTRUCTED THICKNESS FOR FIRST PRELIMINARY TESTS

TEST TRACK STRIP 02 PLAN OF AIR VOIDS AFTER CONSTRUCTION

MIX DESIGN CODE D SPECIFIED COMPACTION HIGH (98 PC) I.E. 5.43 PERCENT AIR VOIDS

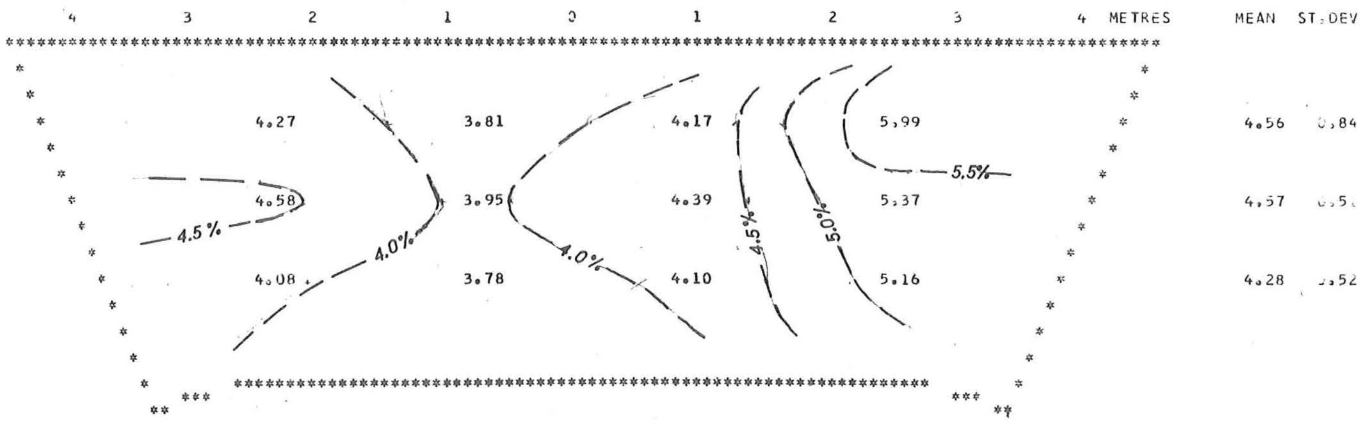


MEANS	3.12	2.92	2.71	2.60
STD.DEVNS.	0.33	0.52	0.33	0.46

MEAN OF ALL VALUES 2.84 %
STANDARD DEVIATION 0.46
NUMBER OF SAMPLES 12.

TEST TRACK STRIP 04 PLAN OF AIR VOIDS AFTER CONSTRUCTION

MIX DESIGN CODE D SPECIFIED COMPACTION LOW(95 PC) I.E. 8.33 PERCENT AIR VOIDS



MEANS	4.31	3.85	4.22	5.51
STD.DEVNS.	0.21	0.07	0.12	0.35

MEAN OF ALL VALUES 4.47 %
STANDARD DEVIATION 0.66
NUMBER OF SAMPLES 12.

Figure 5.5 VARIATION IN INITIAL AIR VOIDS FOR FIRST PRELIMINARY TESTS

TABLE 5-III

DENSIFICATION DATA FROM FIRST PRELIMINARY TESTS

Traffic (passes) C.P. 620 kN/m^2	No. Cores per Value	Air Void Content (%)				Mean Air Voids (%)
TEST STRIP 02						
0	4	3.12	2.92	2.71	2.60	2.84
270, cold	3	2.70	2.30	2.32	2.25	2.39
420, cold) 730, hot)	4	1.15	1.16	1.39	1.75	1.38
TEST STRIP 04						
0	4	4.31	3.85	4.22	5.51	4.47
270, cold	2	3.54	3.12	3.01	4.72	3.60
420, cold) 730, hot)	4	1.83	1.54	1.43	1.77	1.64

5.3.4 (cont.)

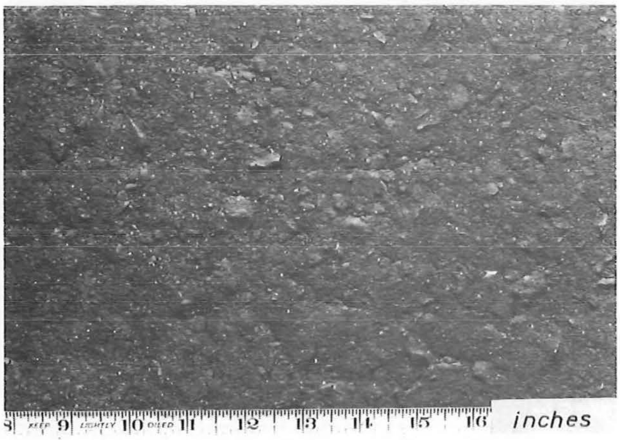
strips. The photographs in Figure 5.6 illustrate this. One effect which is particularly noticeable in photographs (a) and (c) is the erosion of fines from the surface that occurred during the high temperature trafficking. The larger stones protrude from the mix giving a texture with coarse asperities. In photograph (d) another textural effect is noticeable. There is a high incidence of fine cracking that appears to be the result of tensile failure of the binder film. This is indicative of a great deal of instability and movement in the mix under such severe loading conditions. The significant horizontal deformation indicated by the displaced thermocouple slot, ib (e), was caused by the pavement sliding as a mat over the hot concrete base in the direction of travel. This was evidenced by the shear failure existing at the edge of the path, ib (e), and also by the displacement of refilled core holes revealed during removal of the test materials. This is the instance of bond failure due to overheating of the base-surface course interface mentioned earlier. Finally, it should be noted that, although this overall performance could be classified as "failure", the pavement became more than adequately stable again at mild temperatures. The deformities remained, of course, but the pavement had regained satisfactory load-bearing capacity.

The trafficking conditions which caused such a poor performance of the mix are much more severe than those expected in New Zealand. However, the test showed what can happen under extreme conditions and it demonstrated some of the capabilities and drawbacks of the Testing Track facility. Clearly, base heating should be used as little as possible, the pavement should be given the opportunity to develop its strength as it would normally do in the field and the use of a simple profilometer to measure deformations would be desirable.

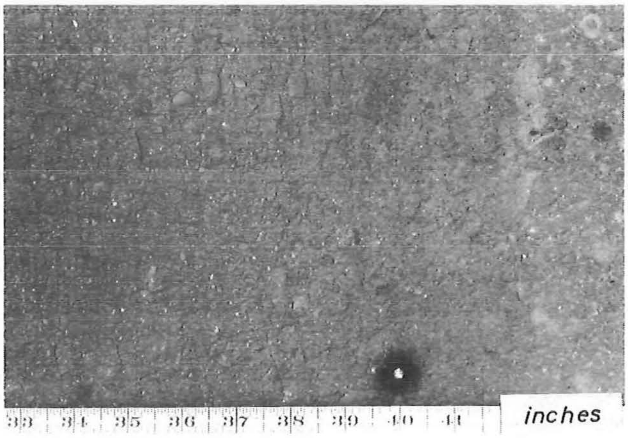
The next two preliminary strips, 01 and 07 tested in October, 1969, provided further experience of construction, a full scale test trial in terms of a load and temperature sequence as mentioned earlier and a chance to perfect measurement techniques. In brief, these strips showed that a great deal of compaction occurs in the first few thousand passes of the Vehicle, i.e. in approximately 6 to 10 hours of

(a) Extensive Rutting, Strip 04

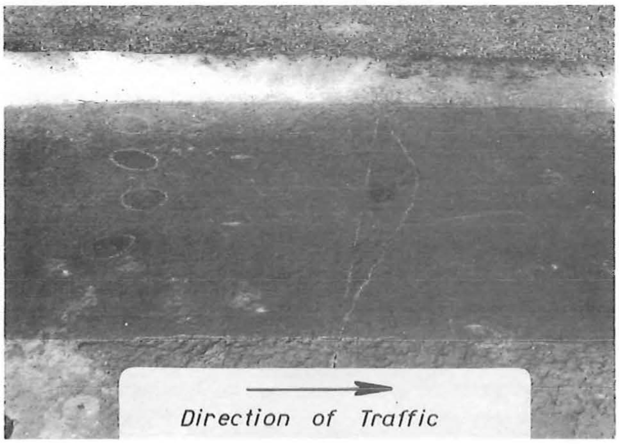
(b) Samples cut from Strip 04



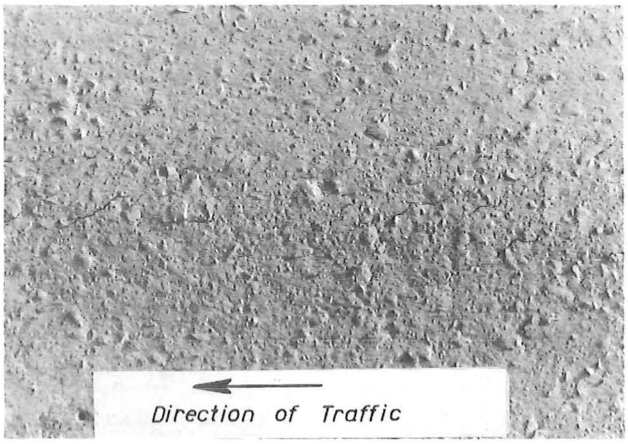
(c) Erosion of Fines from Surface



(d) Micro-cracking - Failure of Binder Film, 02



(e) Horizontal Deformation Caused by Failure of Base Bond, 04



(f) Tensile Cracking along Rut Ridge

N.B. Load C.P. 620 kN/m^2
Temperature 60°C

Figure 5.6 <u>HIGH TEMPERATURE DISTRESS ON PRELIMINARY TEST STRIPS 02 & 04</u>

5.3.4 (cont.)

trafficking. They also demonstrated that the load/temperature sequence does represent increasingly severe conditions. Detailed coverage of these results is included with the main results.

5.4 PROCEDURE FOR TRACK TESTS

5.4.1 Construction

The eight strips for each test series were paved on two successive days doing four alternate strips each day. The octagonal shape of the track caused a certain amount of loss of effective length to each strip through cutting or overlap rolling but this was handled as economically as possible, Figure 5.7. Otherwise the procedures developed during the preliminary tests were generally followed. Photographs of the construction sequence and details for each test strip may be found in Appendices V, VI.

Each truck load was sampled before leaving the mixing plant and Marshall blocks were compacted in the laboratory. The quality control of the loads was consistently good.

A thin coat of bitumen emulsion (180/200 pen.) had been applied to the concrete slabs before paving. This was sufficient to act as a tack coat, but was not thick enough to cause lubrication as happened on one of the preliminary strips.

Compaction proceeded well with only one or two blemishes occurring in the whole operation. The only significant blemish was a tear caused by the steel roller on Strip 12, but this was healed by further pneumatic rolling. Construction densities departed from the specified densities reducing the desired range, but sufficient range was obtained to produce meaningful results, cf. Appendix VI, Figures VI. 1-18.

5.4.2 Temperature Control

The specified middepth pavement temperature for each testing stage was achieved as much as possible by natural solar heating alone. This ensured that the temperature gradient over the depth of the surface course would be both realistic and of the correct sense. The heating elements were used as a fine control to bring the temperature to within one or two Celsius degrees of that specified and, if need be, to keep it there. An example of the temperature control is included

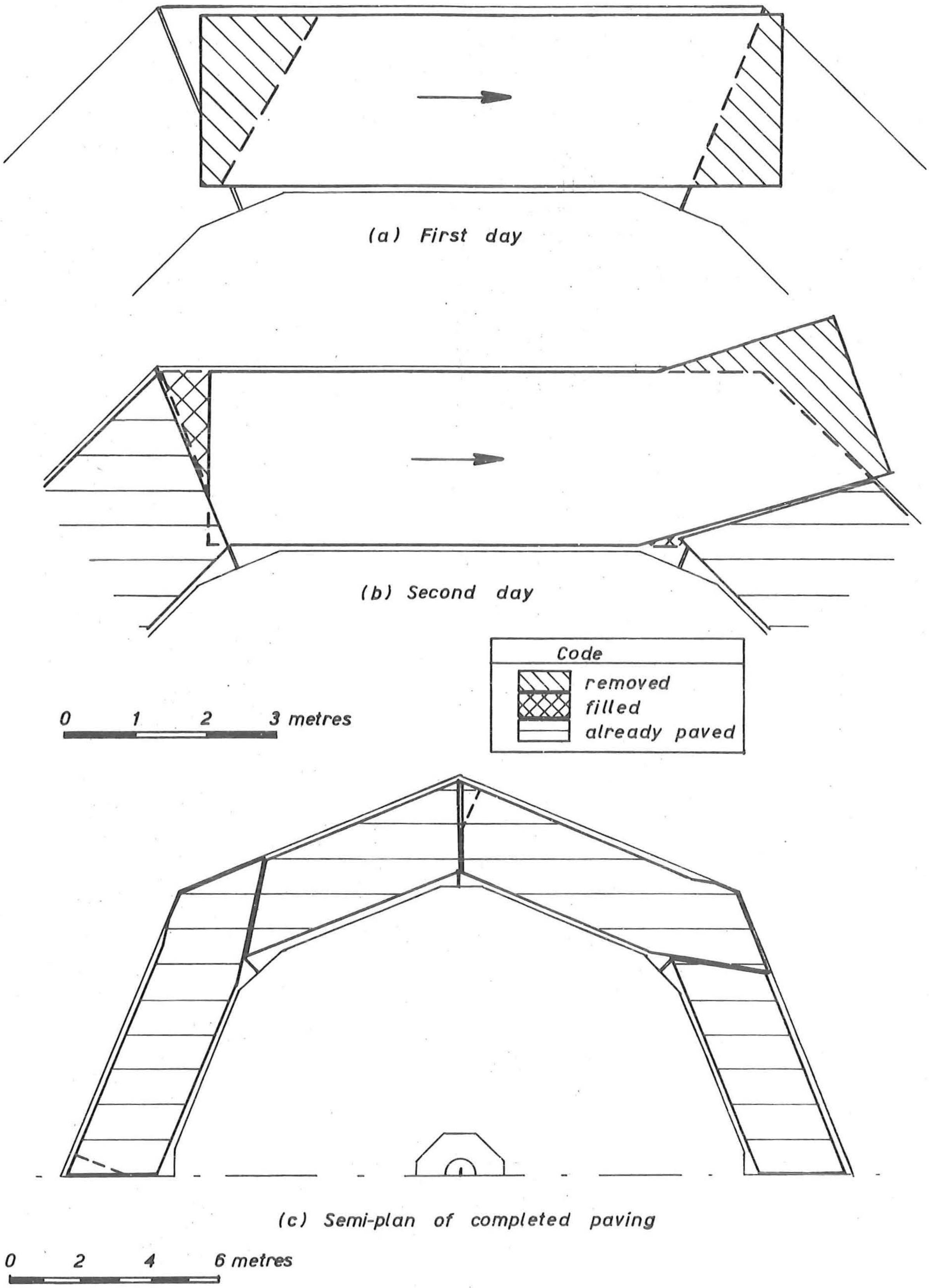


Figure 5.7 PAVING LAYOUT AT THE TESTING TRACK

5.4.2 (cont.)

in Appendix V. The elements were also used on some occasions to extend the duration of each constant temperature period by one or two hours by pre-heating the strip for a few hours several hours before testing was due to begin. The gap of some hours after the heating was switched off gave the base temperature time to decrease before testing began.

Three thermocouples were placed in each test strip. In Test Series 1 these were placed in the centre of the path at the surface, middepth and base of the surface course. In Test Series 2, they were placed in the immediate vicinity of the strain gauges and all at middepth.

Because each test series was conducted during a summer-autumn season, the $25^{\circ}C$ stages were done on cool overcast days or at night when pavement temperatures might remain steady for several hours at a time. The $40^{\circ}C$ stages had to be conducted on fine hot days and then only for a maximum of about four hours a day. Hence each of the four stages of testing, which required approximately 35 hours (25,000 vehicle passes) of trafficking, extended over at least a week for a cool stage and generally more than two weeks for a warm stage. This is the main reason why each test series lasted about three or four months: at times during a warm stage several days would pass before sufficiently fine weather occurred for one day's testing.

5.4.3 Traffic Regulation

Following the general suppositions behind the testing procedure, q.v. 5.1.1(c) that a stable state might be reached for each stage of load/temperature combination, and also the experience of rapid compaction in the preliminary tests, a decision had to be made on the amount of trafficking for each stage. Most of the compaction in the second pair of preliminary test strips seemed to have occurred after a total of 5000 vehicle passes using a similar load/temperature sequence to that in the main test series. Hence a figure of 10,000 vehicle passes for each stage seemed quite large enough to ensure the achievement of stable state density conditions. To check this contention it was decided to make density checks at about one quarter to one third of the way through the stage. After

5.4.3 (cont.)

Test Series 1 was completed some doubt on the truth of that contention was cast by an unexpectedly large increase in voids in a couple of strips over the final portion of the high temperature stages. This was possibly due to the inherent variation in measurements, but nevertheless the trafficking period was extended to 25,000 vehicle passes for Series 2. This matter is discussed more quantitatively in Chapter Six.

The terminology used to describe test trafficking is as follows. The "total trafficking" for a Test "Series" was conducted in a number of "Stages" each representing a different combination of loading conditions. Measurements were made at intervals defined as "substages" during each Stage. The volume of traffic quoted is in terms of vehicle passes, there are two vehicle passes for each full revolution of the Testing Vehicles.

The details of Test Traffic are tabulated in Table 5-IV.

After rutting deformation occurred in the preliminary tests and in Test Series 1 it was realised that the lateral distribution of trafficking was not as realistic as assumed and the mathematical analysis summarised in Appendix Figure I.7 was made. The addition of the 0.15 m section to one vehicle arm produced the more realistic distribution of Figure 4.1 for Test Series 2.

Once the cant of the vehicles had been adjusted to equalise the wheel loads it was fixed by a lock nut for the duration of all further testing, Appendix I. The concrete ballast blocks on each vehicle were moved using a truck crane. Tyre inflation pressures were checked regularly during each test.

5.4.4 Core Sampling

After substages of test trafficking core samples were cut from each pavement strip. The maximum number of cores cut from each strip was determined by the amount of pavement available and the minimum by the amount of variation occurring between samples from the same strip. The procedure adopted was to cut one line of four cores from near the centre of the strip at the intermediate point of the stage, and then either two or three further lines of four cores from the strip at the end of each stage. This meant a total of twelve or sixteen cores from each strip for each stage

TABLE 5-IV

RECORD OF TESTING CONDITIONS AND TRAFFIC FOR ALL TRACK TEST SERIES

TESTING STAGE	LOAD (C.P.kN/m^2)	TEMPERATURE (^oC)	TRAFFIC SUBSTAGE (passes) 1	2	3	TOTAL
SERIES 0 (Strips 01, 07)						
1	280	15	500	1500	-	2000
2	280	45	1400	-	-	1400
3	620	20	1300	-	-	1300
4	620	45	1000	-	-	1000
SERIES 1						
1	280	15	2400	3600	-	6000
2	280	40	2000	4000	5000	11000
3	620	25	3000	5000	-	8000
4	620	40	3000	3000	5000	11000
SERIES 2						
1	280	25	15000	12500	-	27500
2	280	40	7500	12700	-	20200
3	620	25	16150	14900	-	31000
4	620	40	16150	15600	-	31750
4	620	25	-	-	1750	1750
5	620	50	12700	-	-	12700

5.4.4 (cont.)

(the number depended on the length of the strip): 112 cores for each stage, approximately 500 cores for each test series, and a total for the track of about 1300 cores. Each new line of cores was taken at a point 200 mm from the last row against the direction of vehicle travel: this avoided any local impact effects from the vehicle striking an adjacent core. The technique of core sampling is described in Appendix V.

Core samples were preferred for density measurements for several reasons. Density measurements by water displacement could be made to a high degree of accuracy and also breaking tests could be conducted to follow the trend of the "stability" of core samples. This latter is also the reason for preferring 100 mm to 150 mm for the diameter of the core samples. Furthermore, the pavement thickness could be measured from the height of the cores. Even though this process involved a great deal of labour, the additional information was deemed worthwhile. The use of a nuclear densimeter was considered but the cost, lower level of accuracy and limited data obtained ruled it out.

The principal disadvantage of this technique of assessing the pavement's properties lies in the destructive nature of the tests. Measurements at succeeding traffic stages cannot be made on the same sample but must be made on a sample from a nearby location. This opens the measurements to variations inherent in the material as well as to miscellaneous variations in testing conditions, e.g. the particular aggregate gradation, the particular thickness, and the particular temperature conditions because all these may vary to a small extent from location to location.

5.4.5 Measurements on Core Samples

Core samples were measured for density and thickness, and defects in the shape of the core were noted in a code which included the severity of the defect. These defects were departures from the ideal rectangular cylindrical shape caused during cutting and removal of the sample.

Assessment of the increasing strength or resistance to deformation of the material as compaction proceeded was an important concern. It was important because it is clearly a more direct

5.4.5 (cont.)

indicator of reaction to imposed stresses than is density and yet it is so difficult to measure realistically. As was discussed in section 2.3.1, it is important that a "stability" test should replicate the in-situ production of resisting stresses through developing strains and yet few tests do. Even with a theoretical basis to the test, the material may not behave as assumed if relative size factors invalidate the theory. Throughout Chapters Two and Three the behaviour of the material and the nature of the imposed stresses were discussed in detail. The most satisfactory test basis appeared to be linear viscoelastic theory but the laboratory work involved in applying this to the large numbers of samples involved was beyond the resources of this project in the time available during a testing programme. The Marshall breaking test by comparison was simple to perform and, despite the blatant empiricism of the test, it has been shown in section 2.3 to provide a reasonable measure of "stability" when correctly interpreted.

On this basis the Marshall test was selected as the means of following the increase of strength occurring as a result of compaction. It is regarded however more as a qualitative than a quantitative indicator.

Some matters of experimental method required consideration.

(a) Core Structure

The particle structure developed in the field should be preserved because it strongly influences the "stability" of the specimen. For this reason, it was desirable to break the cores as they were rather than to remould them as many investigators have done. This approach places a strong demand on cutting technique in order that the specimens might be "true" in shape, since defects in shape can cause large errors in measurement. This demand was met to a satisfactorily high degree.

(b) Testing Temperature

It was observed during preliminary investigations that the "stability" of such core samples was much lower than the corresponding "Marshall stability" and the reasons for this have been discussed in section 2.3.2. Because of this some of the stabilities measured

5.4.5 (cont.)

were very low giving rise to sources of considerable measurement error in the standard Marshall test and hence the testing temperature was dropped to $50^{o}C$. Pignataro[110] presented a reasonable experimental basis for a simple semilogarithmic empirical relationship between Marshall stability at any test temperature and the standard at $60^{o}C$. The author also considers that there is corrobative evidence from the viscoelastic study mentioned in section 2.3.4 to substantiate this basic kind of relationship. Pignataro expresses the relationship for the effect of test temperature on stability as follows:

$$S_T = 1.567^{\left(\frac{60-T}{10}\right)} \cdot S_{60} \qquad 5(1)$$

where S = stability

T = test temperature, ^{o}C.

This is for a dense-graded mixture with an asphalt binder in the penetration range of 76 - 108 (0.1 mm) and for test temperatures between $30^{o}C$ and $80^{o}C$. The relationship is nearly independent of asphalt content between 5% and 7%. The 95% confidence deviation appears to be of the order of 5% or less and the relationship compares well with data obtained by other investigators. The relationship suggests the following stability ratios for the temperatures involved in this project:

$$\begin{aligned} S_{50} &= 1.567 \cdot S_{60} \\ S_{40} &= 2.456 \cdot S_{60} = 1.567 \cdot S_{50} \\ S_{25} &= 4.814 \cdot S_{60} = 3.073 \cdot S_{50} = 1.961 \cdot S_{40} \end{aligned} \qquad 5(2)$$

The effect of temperature on flow value was not so clearly defined but the author derived the following approximate relationship from Pignataro's data for two asphalts of 76 and 108 penetration grade for test temperatures between $45^{o}C$ and $70^{o}C$.

$$F_T = 1.14^{\left(\frac{60-T}{10}\right)} \cdot F_{60} \qquad 5(3)$$

$$\text{i.e. } \begin{aligned} F_{50} &= 1.14 \cdot F_{60} \\ F_{40} &= 1.30 \cdot F_{60} = 1.14 \cdot F_{50} \end{aligned} \qquad 5(4)$$

5.4.5 (cont.)

While these relationships are not considered to be completely reliable nor necessarily highly accurate, they do give a good relative measure by which to assess results.

(c) Thickness Correction

One final point of experimental method to be considered was the correction factors to be employed for thickness variation. On this matter, Lee[111] made a thorough study at Iowa State University. He found a highly significant correlation between thickness and stability that held for a variety of mixes over a thickness range of 12 to 82 mm. This takes the linear form of:

$$S = S_o \, (at - b) \qquad 5(5)$$

where S = stability at thickness t (mm)

S_o = stability at standard thickness (63.5 mm)

a, b are constants.

The constants a and b varied somewhat for various mixes, especially if remoulded cores were included in the analysis, but "a" was generally in the range 0.07 - 0.31, average 0.20. The constants derived from data from the U.S. Corps of Engineers revealed constants of 0.0232 and 0.48 respectively, and the correction factors quoted by Jackson and Brien[112] reveal constants of 0.0249 and 0.58 respectively. The Iowa figures were preferred in this case because of the thoroughness of the analysis and because it covered a sufficient thickness range. A similar relationship exists for flow correction (The Corps of Engineers maintain that such a correction is unnecessary):

$$F = F_o \, (0.0102\ t + 0.35) \qquad 5(6)$$

where t = thickness (mm)

F = flow (0.1 mm)

However, this seemed to place rather more emphasis on thickness than the author's data indicated and so this correction was modified slightly. The correction factors finally adopted were:

$$S = S_o \, (0.0185\ t + 0.20) \qquad 5(7)$$

$$F = F_o \, (0.0090\ t + 0.43) \qquad 5(8)$$

5.4.5 (cont.)

All of these measurements were made on each core sample and, along with the location of the sample and the stage of trafficking, were recorded on computer cards in the format described in Appendix V, one card per sample. This facilitated handling of the large amount of data involved.

5.4.6 Profile Measurement

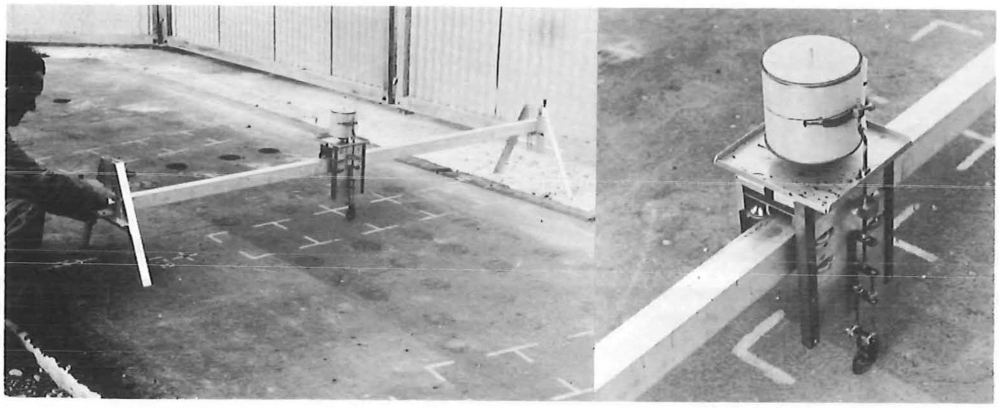
The occurrence of deformations as large as those in the preliminary tests had not been expected and the change in lateral depth profile had been expected to be very small. However, the former experience prompted the design and construction of the simple profilometer shown in Figure 5.8. The profilometer is made from aluminium alloy for lightness. It consists of a rotating chart drum driven by a gear from an axle of the carriage and the pen is actuated by the rise and fall of the travelling wheel. The height dimension is reproduced full scale on the chart but the horizontal dimension is reduced by a factor of 3.75. It is now thought that the height scale should have been increased.

The profiles were regularly recorded over three fixed paths on each strip as shown in Figure 5.9. Each path was designated by a three digit code: e.g. 173, meaning Test Series 1, Strip 7, path 3. Reference points for each path consisted of small metallic discs 8 mm in diameter glued to the pavement 0.30 m outside the trafficked path.

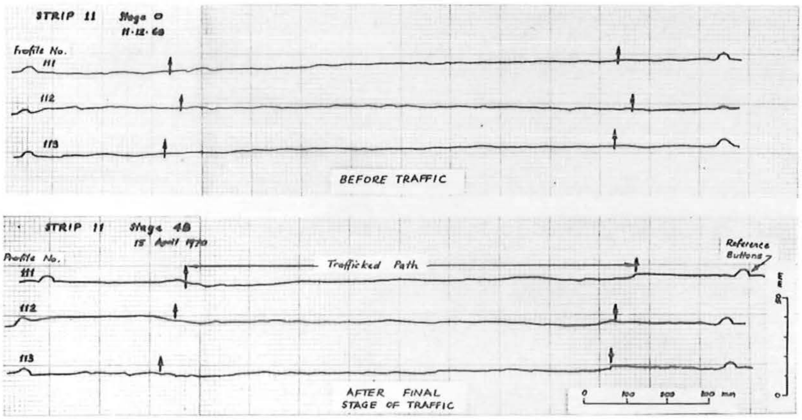
5.4.7 Photographic Record

A photographic record was kept of the surface texture of the test strips. This was used to detect any change such as flushing, microcracking or erosion of the surface. Grid patterns were painted on the surface of each strip along the line of the profile paths so that these areas were kept undisturbed by core samples, Figure 5.9.

Photographs were taken using a 35 mm camera from a constant height of 0.9 m, covering a field about 0.25 m square. The preferred light was bright, slanting sunlight which accentuated the surface texture.



(a) In Use (b) Chart Drum



(c) Typical Records

Figure 5.8 PROFILOMETER

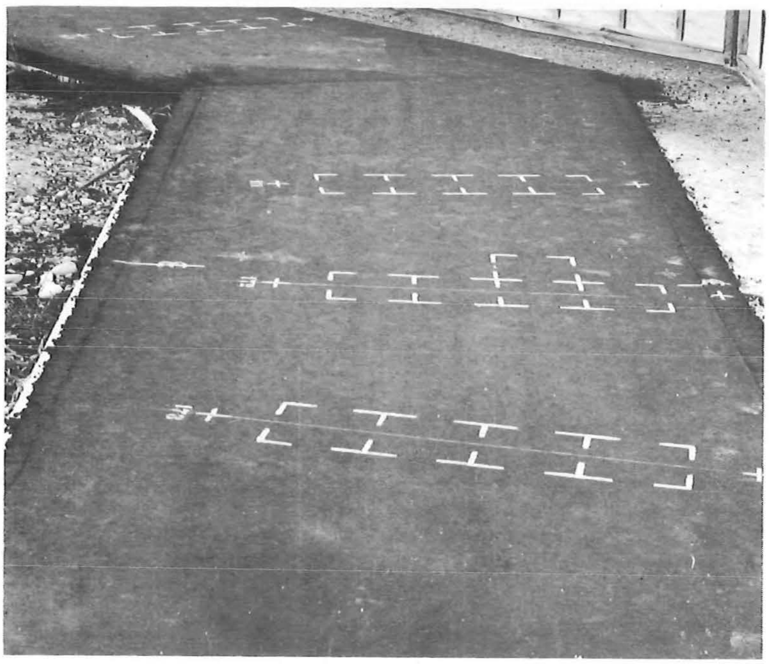


Figure 5.9 LAYOUT OF PROFILE AND PHOTOGRAPHIC REFERENCES

5.4.8 Skid Resistance

The effect trafficking has on the skid resistance of asphalt concrete bears an important relationship to the compacted state of the mix. The condition of flushing is said to severely reduce the skid resistance, and it seems probable that erosion of the surface such as occurred in the Preliminary Tests would also affect the skid resistance qualities of the pavement. Thus a record of skid resistance, as measured by a standard pendulum skid tester, was kept during Test Series 1. It was considered that no great value would be obtained from extending these observations into Test Series 2.

5.4.9 Particle Orientation Analysis

The result of compactive work on asphalt concrete was suggested to be reflected in a re-orientation of aggregate particles as well as a change in density. While the effect was expected to be small, a study of this aspect was begun as a supplement to the main investigations. Samples were taken during Test Series 1 from the trafficked portion of each strip after Stages 2 and 4 and from an untrafficked portion to determine the initial stage. Details of the method of analysis are included in Appendix IV and the results of the data processed to date are discussed in the next chapter.

5.4.10 Strain Measurements

Much of the observational procedure described up to this point is along the empirical lines of earlier studies although the control and simulation of loading and external conditions is far more rigorous than in other studies. More absolute measures of stress and strain conditions have usually foundered in the past because of the difficulty in perfecting a reliable technique. Such an attempt to measure stress or strain conditions were ruled out in the early stages when this project was much smaller but, when the scope increased, the matter was pursued and strain measurements were made throughout Test Series 2. The technique, scope and results of the observations of strain are covered fully in Chapter Seven.

5.5 HIGHWAY TEST AREAS

5.5.1 Main North Road, Christchurch

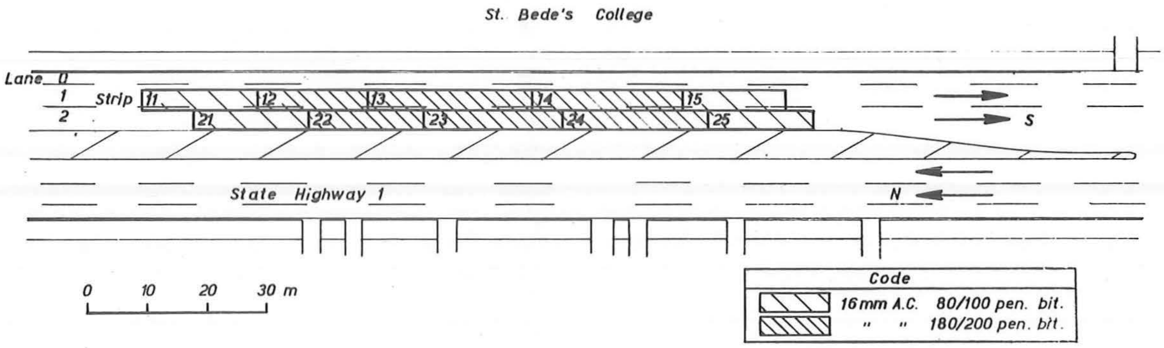
The site is on a long straight section of the south-bound lanes of a four-lane divided highway in a restricted speed area (48 km/h). A right hand turn at the end of the test area is used infrequently. The test area consists of ten strips each constituting the paved area from a single truck load. A plan of the area is shown in Figure 5.10(a), and a view in ib(b). The strips were paved as part of a normal job in February, 1968, at the beginning of this project. This was done under the direction of Mr J. Pollard, one of the supervisors of this project, and the author was present also. The job was a 50 mm thick smoothing overlay to a chip-sealed pavement with a cement-treated base. A thin corrective layer was laid before the overlay.

Two mixes were used, both of standard gradation but with a slightly smaller maximum aggregate size than is usually used in this country: 16 mm instead of 19 mm, Figure 5.3. Two binder hardnesses were employed, 80/100 and 180/200 pen., but the two asphalts were from the same refinery. The mix was produced from a continuous type plant with only average quality control.

Core samples were extracted on a random pattern after construction but, after the lanes had been marked and the highway had been opened to traffic, cores were taken from the wheel paths, 0.9 m in from each lane marking. These were taken at periods of 8 and 21 months.

5.5.2 Christchurch Northern Motorway

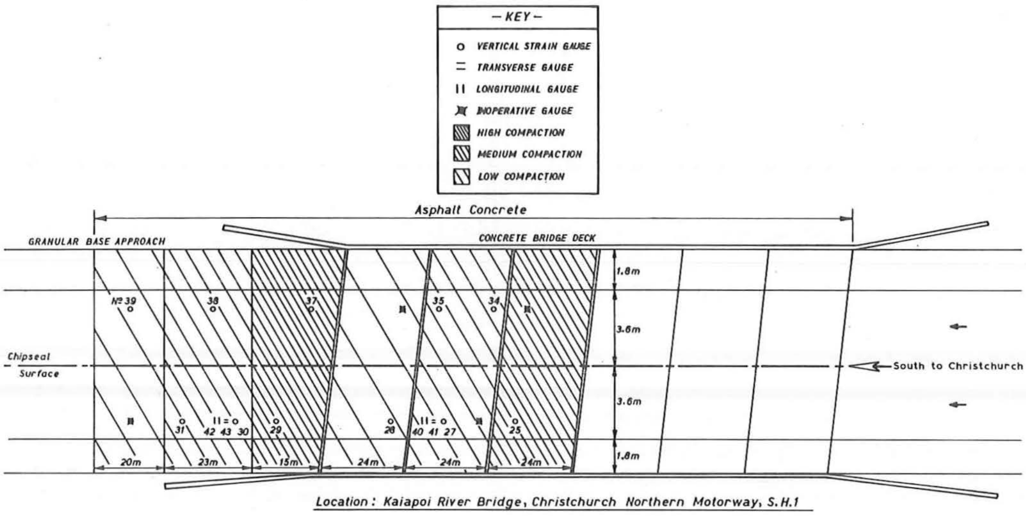
The second highway test area was chosen on the two south-bound lanes of the Christchurch Northern Motorway which has a 96 km/h speed limit. The site consists of a 60 m length on the Kaiapoi River bridge which has a concrete deck, and a 60 m length on the granular-base runoff from the bridge, Figure 5.11. The specific object of this test was to compare the behaviour of the surface course over "rigid" and "flexible" bases under normal traffic. However this granular base was thick and its upper layers stabilised with a bituminous emulsion. In consequence, flexibility was low with a Benkelmann Beam deflection of 0.15 to 0.20 mm.



(a) Plan

(b) View

Figure 5.10 MAIN NORTH ROAD TEST AREA



(a) Plan

(b) View

Figure 5.11 CHRISTCHURCH NORTHERN MOTORWAY TEST AREA

5.5.2 (cont.)

Six full-width strips were laid on the site in December, 1970. A single mix design was used for all strips: again a replica of the Main North Road mix with 80/100 pen. asphalt. However, the supplier did not match the specified grading, Figure 5.3: the mix was more open and had low workability, and the variability was high.

The three strips laid on the bridge deck each ran the length of one 20 m bridge span full-width. The rolling was progressively overlapped so that the first strip received heavy rolling, the second strip medium rolling and the third light rolling. The same was done for the three strips on the bridge run-off. The range of construction voids was disappointingly small however and was virtually masked by the variation from sample to sample.

Nineteen strain gauges were embedded in the strips as indicated on the plan in Figure 5.11(a). They were located in the wheelpaths nearest the edge of the pavement to facilitate a safe connection to recording instruments without excessive cable lengths. Core samples and strain measurements were taken at periods of 0 and 8 months before the project was ended late in 1971.

CHAPTER SIX

INTERPRETATION OF THE PHYSICAL EVIDENCE OF TRAFFIC COMPACTION

The static physical data from the Testing Track and highway studies are analysed employing a statistical smoothing technique by digital computer. A number of indicators of the effect of traffic are considered and the influence of each testing parameter on these indicators is discussed. Correlation is made between the Testing Track and highway studies. The outline of a mechanism by which traffic compaction occurs is formulated.

6.1 TREATMENT OF DATA

6.1.1 The Data and the Aim of Treatment

The data discussed in this chapter include all the measurements made on extracted core samples, measurements of surface profile and of particle orientation, and the photographic record of texture. These are all effects of a permanent or semi-permanent nature. The measurements of transient response of the pavement to a passing wheel load are discussed in the following chapter.

Data from experimental pavement studies are generally subject to a large amount of "random" variation which often obscures the significant trend. This random variation may arise from factors such as the non-homogeneity of the material and slight variations in boundary conditions. In order to isolate the significant trends it is necessary to use a statistical smoothing technique. This is true even for the reasonably controlled conditions of the Pavement Testing Track.

In this case, the significant trends of various indicators of the compactive effect of traffic are required. For the Testing Track data these trends must be analysed separately for each combination of load and temperature conditions, i.e. for each testing Stage. In this way the distinct characteristics of each set of conditions is preserved and the relative effects of load and temperature may be compared. Correlation of these findings with normal highway conditions, which represent a mixture of these loading combinations, will involve consideration of all Stages together.

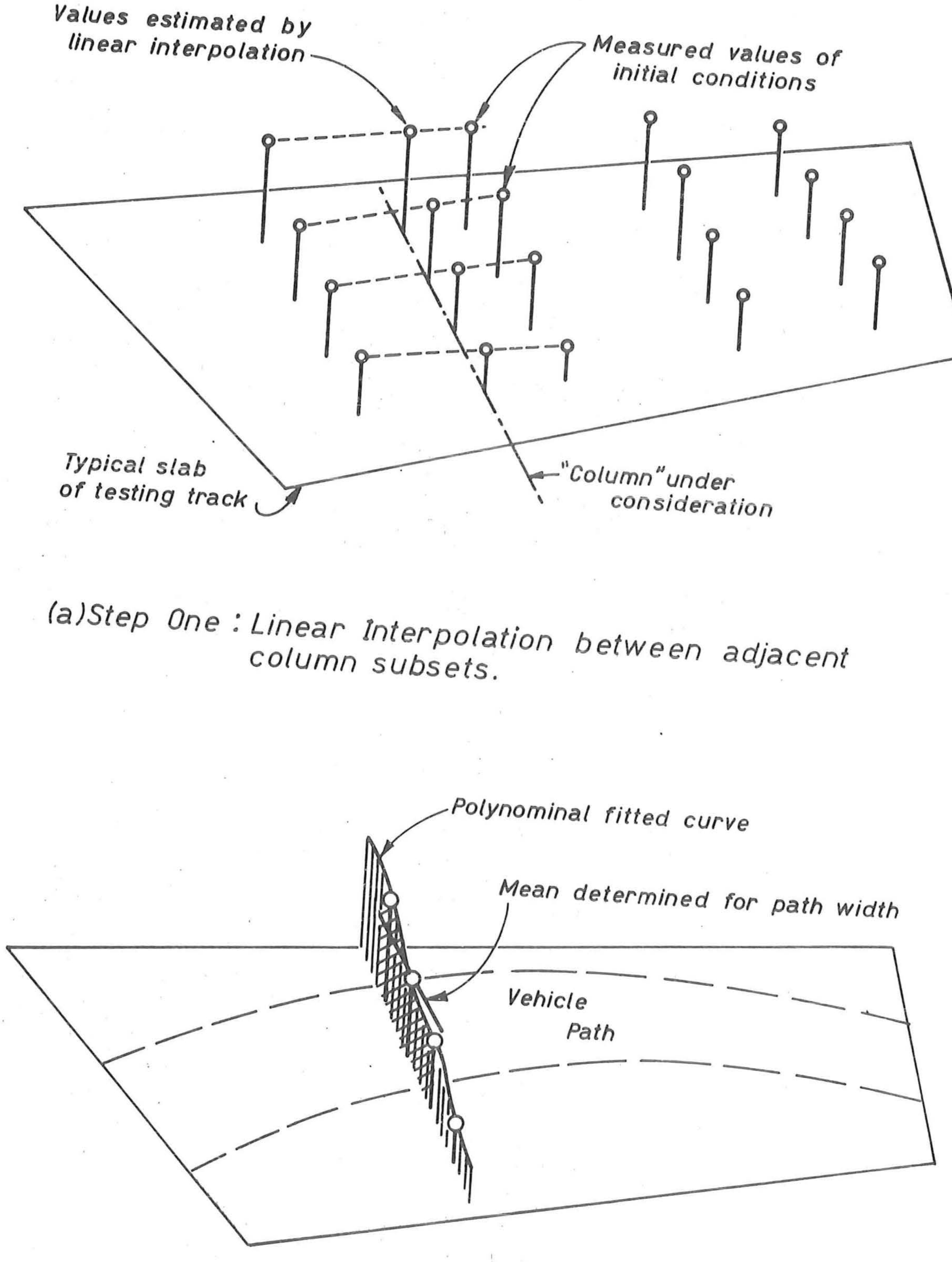
It has been postulated earlier that some kind of "stable state" of pavement properties is attained when the pavement has developed sufficient resistance to the loading conditions so that no further permanent deformations occur. Preliminary tests at the track showed that this occurred after relatively few load applications. Analysis of the data should test this postulate against experimental evidence. At the same time the analysis should define the magnitude of such a stable state or, at worst, if the trend has not levelled off, it should reasonably predict the asymptotic value of the trend.

6.1.2 Abstraction of Densification Data

A program was developed to analyse the Testing Track data on the University's IBM 360/44 digital computer. Details of the program are included in Appendix VII but this section and sections 6.1.3 and 6.1.4 describe its rationale and general functioning.

The program analyses all the data for every loading condition for one strip before analysing the next strip. The initial or construction conditions, designated Stage 0, are read in and stored in a subprogram named "CONDAT" (for Construction Data). This subprogram when called estimates the initial conditions at any location on the strip within the initial data. It does this by simple interpolation in the longitudinal direction followed by fitting a curve by polynominal regression analysis to the values in the transverse direction, see Figure 6.1. The smoothed values occurring over the trafficked path are then averaged to provide the datum conditions. The reason for this process comes from the directional variations observed in the construction data. There are significant variations in the transverse direction, principally with light compaction near the edges, but there are only small variations longitudinally, the direction of rolling. This subprogram is used for estimating the datum conditions for Stage 1 and is also used to establish the datum for a transverse profile of values across the path, a process described later.

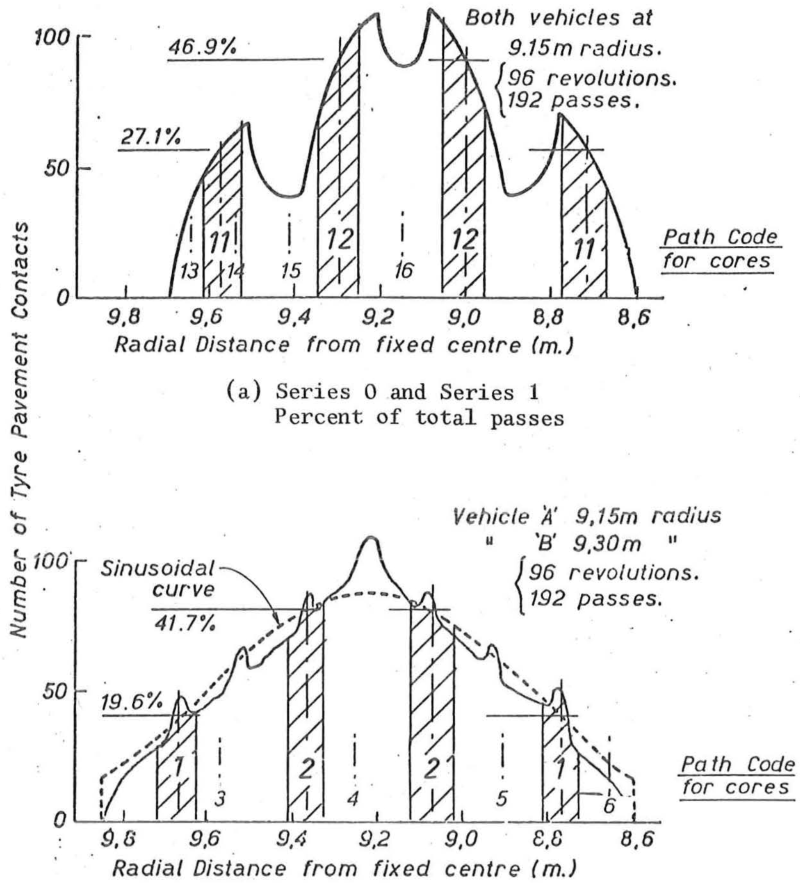
Traffic volumes were measured from zero at the beginning of trafficking for the testing Stage under consideration. At least three basic volumes of traffic are available taking into account the substages of testing: zero, intermediate and final, where the "intermediate" is generally about one third of the "final" and in some cases there are two intermediate volumes. In addition to this it should be noted that the number of tyre-pavement contacts varies transversely across the path, Figure 6.2, cf. 5.4.3, App. I, Figure 4.1. For the symmetrical spacing of the four core samples across the path it can be seen that each one receives only a portion of the total number of load applications. The two outer cores receive approximately 27.1% of the total for Series 0 and 1 and 19.6% for Series 2. Similarly, the two central cores receive 46.9% for Series 0 and 1, and 41.7% for Series 2. Figure 6.2 also demonstrates the code used



(a) Step One : Linear Interpolation between adjacent column subsets.

(b) Step Two : Determine mean from transverse function

Fig. 6.1 ESTIMATION OF INITIAL CONDITIONS FOR THE TESTING TRACK



(a) Series 0 and Series 1
Percent of total passes

(b) Series 2

Figure 6.2 LATERAL LOCATION OF CORES AND PATH CODE USED WITH RESPECT TO TRAFFIC DISTRIBUTION

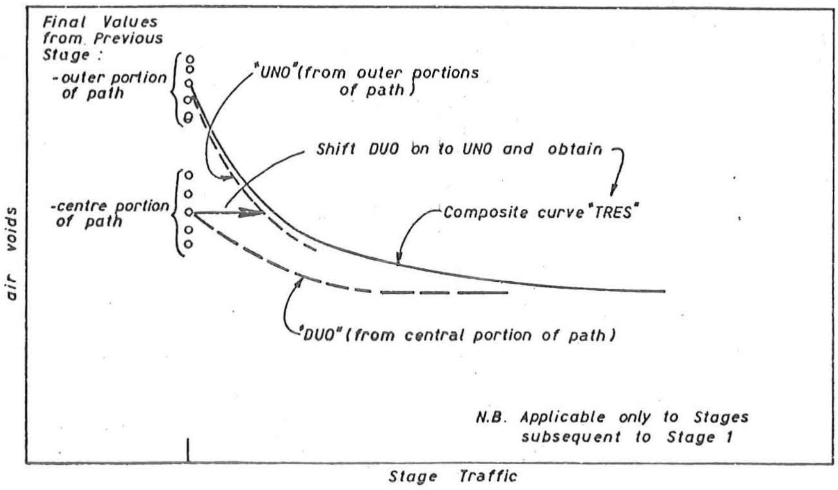


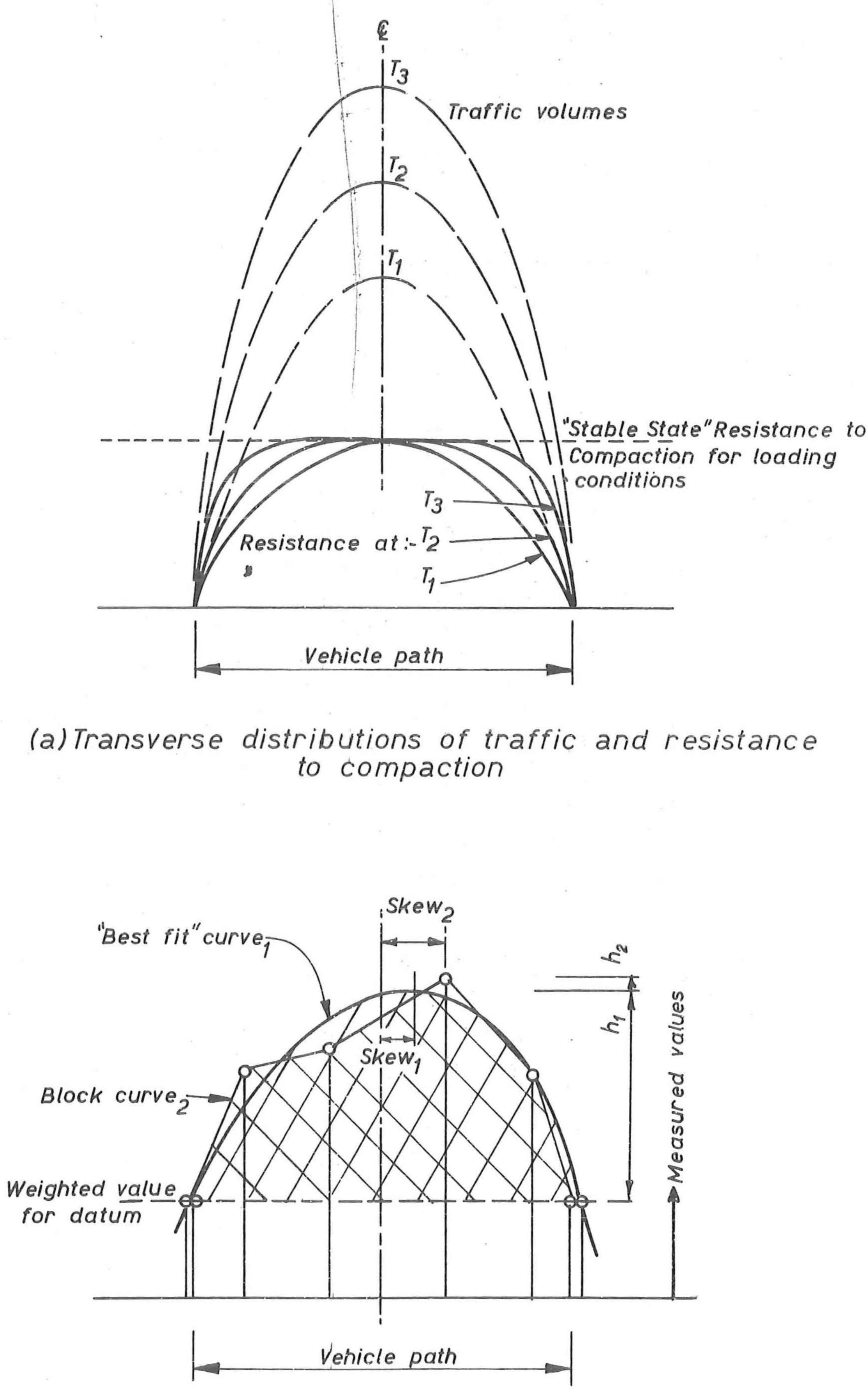
Figure 6.3 SUPERIMPOSITION OF TRAFFIC FUNCTIONS

6.1.2 (cont.)

to designate the location of each core within the trafficked path. On this basis the traffic volumes are computed for each core as the number of actual load applications received by the core. This step is made possible by the adoption of tyre inflation pressures that produce near-rectangular transverse distributions of contact pressure, q.v. 5.1.1(a). This step also adds another level of traffic volume to each substage mentioned earlier so that there are effectively a minimum of five traffic substages per Stage.

It is expected then that the central cores should densify faster than the outer ones. If a stable state is reached, all cores should be at the same density by the end of a Stage, but this should not be presumed because the stable state postulate must be tested by experimental evidence. For Stage 1 of the testing, the pavement is starting with approximately uniform properties and hence every sub-stage volume of traffic contributes to a single curve, Figure 6.3. For subsequent Stages however the initial properties cannot be presumed uniform if a single stable state has not been reached for the previous conditions. Hence the outer portion of the path is analysed separately from the central portion of the path, resulting in the curves "UNO" and "DUO" respectively. These two curves can be combined if initial conditions are assumed to not affect the final outcome of trafficking. The curve "DUO" can be shifted horizontally by an increment of traffic until its initial condition lies on curve "UNO". The two curves should then be superimposed, and the curve "TRES" may be taken to represent the general trend with traffic volume for the particular loading conditions. The curve "TRES" should exhibit a marked flattening in its latter portion if a stable state has been reached.

Yet another check on the attainment of a stable state should come from the transverse profile of indicator across the trafficked path. Figure 6.4 shows the superposition of several traffic profiles at increasing volumes, cf. Figure 6.2, on top of the equivalent profiles of resistance to compaction. After a traffic volume of T1 the centre of the path has just reached a stable resistive strength and after a traffic volume $T_2 > T_1$ a larger proportion of the path width should have attained this stable strength. In other



(a) Transverse distributions of traffic and resistance to compaction

(b) Alternative areas and skew values

Fig. 6.4 ILLUSTRATION OF THE TRANSVERSE "SQUARENESS FUNCTION" POSTULATE

6.1.2 (cont.)

words, the profile of "strength" should be becoming more square in shape than sinusoidal. This property is tested in the subprogram "SQFN" (for Squareness Function) and is presented as an area ratio of the measured curve to the rectangular curve of equal height. The measured curve which is used is either a smoothed curve drawn through the profile points or a block-type curve, whichever gives the greater area, see Figure 6.4(b). The "skew" of the curve is measured as the percentage departure of the peak value from the peak traffic, i.e. from the centre of the path. The datum values used are computed in the subprogram "CONDAT" already mentioned: the estimated initial values at the two extremities of the path are averaged and then given double weight to ensure that the profile curve passes through these two data.

6.1.3 Statistical Smoothing Techniques for Densification Data

Given a selected and treated group of data containing some significant trends but also some less significant deviations from the trends, the task is to smooth the data statistically. Two important properties of such a trend are known and may be assumed from the start. Firstly, the trend may be represented by a "continuous" or "smooth" function rather than a discontinuous one. Secondly, the initial rate of change will be high but it will decrease continuously as traffic volume increases. A third, but less certain, characteristic is that this rate of change should decrease to, or near to, zero and should not reverse.

There were very few techniques to choose from for such a non-linear function. The only general method pertinent to this case and available was a polynomial regression analysis developed as a computer program. The analysis fits an increasingly higher powered polynomial to the data until it obtains a "best fit" determined by a regression performed on the dependent variable. As an aid to the ensuing discussion a typical data set is illustrated in Figure 6.5(a).

The polynomial is formed of positive integer powers so that, when applied to the data set in this form, with traffic volume as the independent variable it could generate an inflexioned curve usually upwardly concave, curve "A" in ib.(b). At powers higher than two it

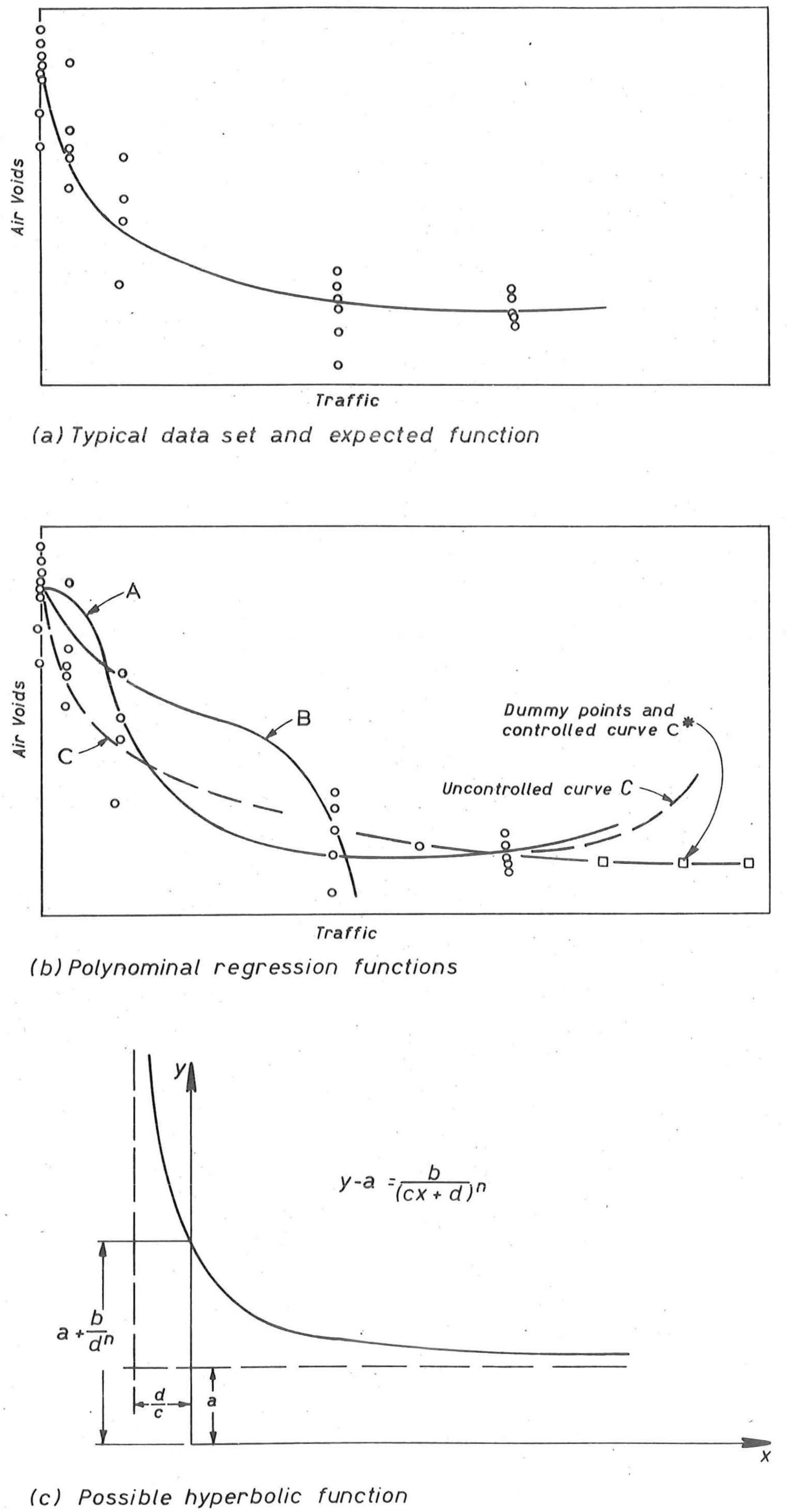


Fig. 6.5 GENERATION OF TRAFFIC CURVES

6.1.3 (cont.)

could begin to oscillate in order to reduce the deviation of scattered data.

An attempt to overcome this was made by reversing the variables so that traffic volume became the dependent variable. However this produced worse results because the deviation of a high point could be infinite for the expected smooth curve. So the analysis bent the curve around to incorporate that point which normally would be excluded, curve "B".

The earlier approach with traffic volume as the independent variable was more promising and so this was resumed. This time the independent variable was reduced to a fractional positive power before being submitted to the analysis, so that the analysis developed a function of the form:

$$y = a_n x^{n/n} + a_{n-1} x^{n-1/n} + \ldots\ldots\ a_1 x^{1/n} + a_o \qquad 6(1)$$

This process yielded satisfactory results, performing the regression in the desired direction and yielding a satisfactory curve of low power, curve "C".

However the author desired to derive a characteristic function for the curve rather than a general polynomial function. He surmised that the asymptotic character of the tail of the curve might indicate a hyperbolic function. If this were so and if the head of the curve could be considered as a curtailed asymptotic function, then the function would be a rectangular hyperbola of the form:

$$y-a = \frac{b}{(cx+d)^n} \qquad 6(2)$$

where a,b,c,d and n are constants defined in Figure 6.5(c). Note that 'a' is the value attributed to the stable state which is being sought. The curve could be treated in this form though it is difficult with four unknown constants. It is easier to transform the components into logarithmic form in order to obtain a linear function of slope n . To do this it is necessary to first know the constants a,c, and d to produce the function:

$$\log (y-a) = -n \log (cx + d) + \log b \qquad 6(3)$$

6.1.3 (cont.)

This can be achieved on the computer by an iterative trial process of choosing an estimate of 'a' and then trying successive values of 'c' and 'd' until a best fit linear function is obtained. The process is repeated for various values of 'a' until the best possible fit is found. Although laborious it is feasible in principle. However, the method could not be implemented because it could not cater for any data scattered beyond the 'a' value. Such data gave rise to a negative argument for a logarithm. Furthermore, in that form the regression analysis would work against low values in favour of high values because of the scale effect involved.

The statistical smoothing technique finally adopted was that of the polynomial regression analysis using fractional positive powers. It was found that the highest power necessary for a satisfactory curve was two. This also reduced the likelihood of an inflexion or reversal of curvature occurring in the curve.

6.1.4 Selection and Weighting Techniques for Densification Data

Even with the data abstracted in the manner described and treated with a statistically smoothing curve-fitting technique, the presence of extraneous variations may significantly disturb the trend. For example, if thickness has an important influence, as has been suggested, then data from a sample whose thickness falls outside the range being considered ought to be excluded. Large thickness variations do occasionally occur near the edge of a strip, near the end of a strip with transition to the neighbouring one, and due to irregularities in the level of the concrete base. Hence data is excluded if the core thickness is outside the extremes of those measured from the initial construction stage. While this may seem a very loose restriction stricter control could not be exercised without excluding too much data, cf. 5.3.3.

Another loose selection process incorporated in the analysis was that the air voids during the first stage of traffic should not exceed 90% of the maximum single value of initial voids. This was included specifically to exclude one or two areas where mix segregation was thought to have occurred at the end of a paving run during construction.

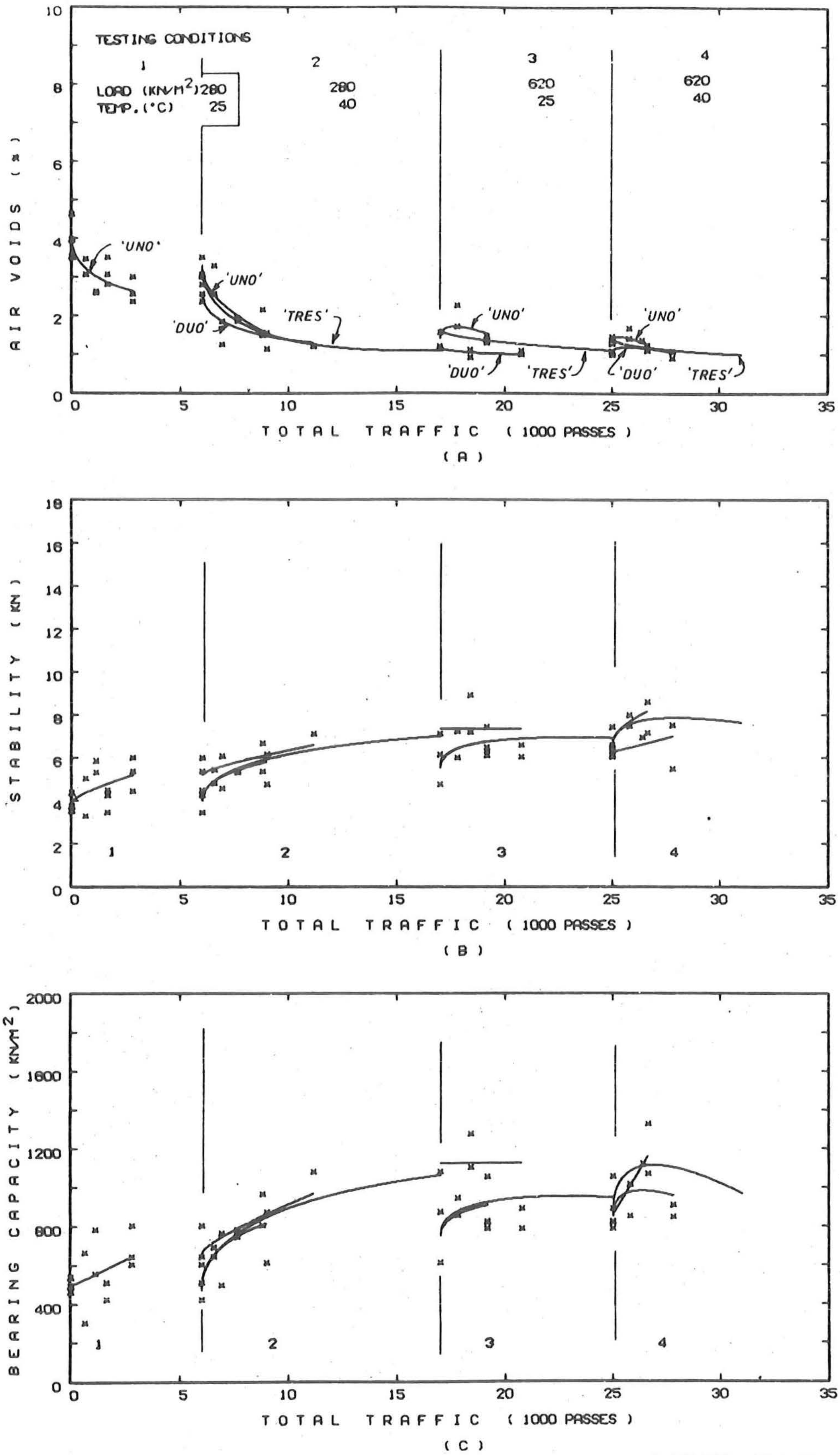
6.1.4 (cont.)

A more important selection process was on the basis of sample shape. Clearly a sample with a distorted shape may produce doubtful results. On the basis of the shape and severity codes, q.v. 5.4.5, any badly distorted sample and any sample with a distorted shape that would significantly affect measurement of various indicators, such as stability, was excluded.

The sample size for each strip is generally small ranging from 8 to 20 values in total and less if some are excluded. Thus a single large deviation in the region of maximum traffic can considerably divert the curve from its anticipated low or zero gradient. There are few logical grounds for excluding such a value in a small set but it does seem reasonable to give it a lesser weighting. It is difficult to develop a simple test to determine which are these values. This is because the curve is not constrained to flatten in gradient or to prevent it reversing its gradient. Hence it would devalue reliable values as often as unreliable values. Thus a curve "flattening" device was incorporated in the analysis. It consisted of adding three arbitrary points into the array of variables beyond the range of maximum effective traffic (M.E.T.) at 120, 140 and 160% of the M.E.T. and giving them double weight, see Figure 6.5(b). This device is a valuable technique in handling the polynomial regression analysis because it controls the curve immediately beyond the data region, curve C*. The effect the device may have on weighting the data is secondary. The data in the region of M.E.T. generally has low scatter anyway and so most of this data may be regarded as reliable. The value attributed to these dummy points is the mean of the best two-thirds of the M.E.T. data, thus providing some weighting against a scattered value.

6.1.5 Presentation of Densification Data

The relationship between traffic volume and physical core properties were plotted by an IBM 1620/1627 computer and plotter from the output of the main program for the Testing Track data. These detailed graphs are included in Appendix VI, Figures VI. 19-38. One of these is reproduced in Figure 6.6 for description purposes. The relationships and the data for air voids, stability and bearing capacity



THICKNESS- 52.1 MM, σ= 5.3
ASPHALT - 6.10% 180/200 PEN
AGGREGATE- 16.0 MM MAX SIZE

Figure 6.6 TYPICAL GRAPHS RESULTING FROM COMPUTER ANALYSIS - TRACK STRIP 16

6.1.5 (cont.)

are given in separate graphs for each strip. Each graph is composed of a series of discontinuous curves. These are the sets of three curves UNO, DUO and TRES generated in the analysis (6.1.2 para. 4) for each combination of loading conditions, described as a "Stage" of testing. Each set of curves is the result of a different loading condition and hence these sets are not linked and must be regarded separately.

For example, examination of Figure 6.6(a) shows that the pavement densifies to 2.6% air voids under trafficking at 25°C by a wheel load with 280 kN/m^2 contact pressure ("Testing Condition 1"). Similarly, at a pavement temperature of 40°C, the same wheel load ("Testing Condition 2") densifies the pavement further to 1.2% air voids, according to the composite curve TRES which is taken as the master curve. All the graphs are to be interpreted in this manner. Hence the corresponding maximum values of stability and bearing capacity for Testing Conditions 2 are 7.2 kN and 1040 kN/m^2 respectively.

It may be noticed that the beginning of a curve in Stages 2, 3 or 4 does not always correspond to the tail of the preceding curve. This occurs because the initial value for the curve under consideration is the final condition of the lightly trafficked outer portion of the path. The tail of the preceding curve is the statistically predicted stable state for the existing conditions and is likely to be 'greater' than the final state of a lightly trafficked area. The analysis shows such an effect, it does not predetermine it.

This method of presentation allows a ready assessment of the comparative effects of the various loading conditions. The results may be adapted to any set of load and climatic conditions by a combination of these testing Stages.

In this chapter only the master curves obtained from the data for each strip are used. To simplify discussion of the results the curves from various strips are collected in groups in Figures 6.7 - 6.14 and a tabulated summary of the computer output is given in Table VI-I. The analysed data may be referred to in Appendix Figures VI. 19-38 if desired.

These pages are fold-out sheets of Figures 6.7 to 6.20.

For binding purposes they have been placed at the end of this chapter after p.167.

6.1.5 (cont.)

Data from the highway studies on the Main North Road and the Northern Motorway were processed manually and are presented in detail in Appendix VI. Only the processed curves are presented in this chapter for discussion, Figures 6.15, 16.

6.1.6 Abstraction of Data of Permanent Deformation from Transverse Surface Profiles

The transverse profiles of permanent deformation recorded by the profilometer were measured by superimposing the data chart on the chart representing initial conditions. The profiles of net permanent deformation so measured were transferred to a graph with a vertical exaggeration of 60:1, q.v. Appendix Figures VI. 39-58. They were quantified using a planimeter to obtain the average net permanent deformation, the standard deviation from this average (a measure of rutting distortion within the path), and the maximum rut depth. The quantified characteristics are tabulated in Appendix Table VI-II and depicted collectively here in Figures 6.17, 18. Possible error in an individual profile is estimated to be $\pm$ 0.5-1.0 mm and in the averaged, quantified characteristics approximately $\pm$ 0.5 mm.

6.1.7 Sequence of Discussion on the Results

The selected developed results are discussed in the following sections. Firstly, the information obtained from the indicators of density and air voids, stability and flow, bearing capacity, squareness functions, transverse surface profiles, surface texture and skid resistance is discussed. Then the effects of imposed conditions and mix design are considered in the terms of these indicators and the results of a pilot study on particle orientation is presented. Finally, a thesis on the mechanism of deformation is formulated to account for these observations.

6.2 DISCUSSION OF THE INFORMATION GAINED FROM THE INDICATORS

6.2.1 Density and Air Voids

Bulk density and the consequent parameter of air voids has long been the simplest and most popular indicator of the condition of an asphalt concrete surface course. Bearing in mind the distinction

6.2.1 (cont.)

made earlier between density and structure, and the need to study the effect of imposed stresses by compatible parameters, namely stress and strain, how valuable are density and air voids as indicators?

The magnitude of air voids is dependent on the density assumed to represent the zero air voids (Z.A.V.) condition. While the magnitude is open to errors of the order of $\pm$ 1 air voids units, the relative trend of the air voids value will be much more accurate since all values for any one mix design have been calculated on the basis of the same Z.A.V. density.

A study of the results summarised in Figures 6.7 - 14 yields the following observations on the value of air voids as a parameter.

(i) There is a significant trend in all the data of decreasing air voids under the repeated loading action of traffic for each testing condition, so long as the pavement is not severely distorted. However the raw data in Appendix VI, Figures VI. 19-38 show that there is sometimes a large amount of scatter amongst individual measurements so that there is a possibility of apparent contradictions to such a trend. This contradiction is definitely not significant when the data is treated statistically as described in the previous section, but it may appear to be extraordinary in cases such as Strips 14 and 15. There, the air voids values attained under Testing Conditions 3 appear to be slightly higher than those attained under Testing Conditions 2, but this is not significant. The qualification on the shape of the pavement mentioned in the initial statement of this clause is amplified in Clause (iii).

(ii) The amount of scatter in the air voids value decreases under traffic. This may be gauged quantitatively from the 95% confidence limits quoted in Appendix Table VI-I.

At worst, these limits initially are $\pm$ 1-2.5 a.v.u. and eventually decreasing to $\pm$ 0.3 - 0.8 a.v.u. Strip 17 has the most consistently low scatter (0.57 - 0.24 a.v.u.) and Strip 13 shows one of the most marked decreases in scatter (2.50 - 0.83 a.v.u.). However, the scatter for some strips such as 04, 01, and 22 increases in the latter stages and for some other strips such as 12, 26 and 28

6.2.1 (cont.)

it remains relatively high throughout. There are three possible explanations for these anomalies. Firstly, a wide scatter, as in the latter anomaly, will be preserved if the initial air voids condition does influence the final condition. This is discussed fully in Section 6.4 but indications are that it is a contributing factor under the less severe testing conditions. Secondly, it may be the result of thermal cycling. This was an explanation considered by Epps et al[8] but found to be not significant. This was not checked by the author. The third explanation is considered in the following clause.

(iii) An increase in air voids value or scatter may be caused by distortion of the pavement. If the pavement becomes severely distorted in shape it is said to have become unstable under those loading conditions. Such instability arises when insufficient resistance is developed to support the imposed load without excessive deformation. Such excessive deformation coupled with low lateral support gives rise to significant horizontal stresses, lateral movement and upthrust in areas adjacent to the loaded area. This is far from the normal condition and would probably cause a decrease in density and an increase in air voids. This is borne out by strips 04, 01 and 22, Table VI-I. However, some other strips suffering distortion such as 02, 14, 16 and 24 do not show as strong a reversal influence on air voids.

(iv) The magnitude of the air voids value at stable state is generally equal to or lower than the design air voids value after Stage 2. However, if the air voids of the laboratory-compacted samples taken from the plant during production are used as a measure it can be seen that nearly all of the strips reached the predicted final air void content only after Stage 4 or 5 of testing. The mixes on the Main North Road achieved an air voids value slightly lower than predicted. These observations imply (a) that the 75-blow Marshall density appears to be a satisfactory guide to the final stable state density and (b) that the Testing Track simulation of trafficking appears to be producing realistic results. These implications are discussed in detail in Sections 6.3.3 and 6.7. Some discrepancies are nevertheless to be expected, both because of the empirical nature of the measure and because of production variations. The quality control of the

6.2.1 (cont.)

batch plant used for the track mixes was better than ± 1 kg in 30 kg of bitumen in a 500 kg batch. This produces expected limits of ± 0.5% on the calculated Z.A.V. density, that is ± 0.5 a.v.u. on the design air voids.

(v) The distinction between density and structure is unlikely to be very significant in the surface course situation. The structure is probably highly developed during construction rolling and traffic action is likely to have only a small influence on that structure. Hence the changes in density and air voids are probably a good relative indicator of strength in this case. Structure may be significant, however, when construction densities differ for the same mix.

6.2.2 Stability and Flow

The parameter of Marshall stability, despite its empiricism and the cautions mentioned earlier, is a more direct measure of pavement strength than density. Is it a more valuable indicator in this case?

(i) The stability of extracted cores increases under the action of traffic for each testing condition, provided the pavement is not badly distorted in shape. While the trend is very similar to the increase in density it is not always comparable. On a few occasions there is an increase in stability when there is virtually no increase in density, e.g. Strips 23 and 26, Stage 3, and in one case, Strip 13, Stage 4, there was negligible increase in stability for a significant increase in density. However, these departures from the general trend are few and are probably explained by the high incidence of scatter discussed in the next clause.

(ii) The stability data exhibits a high degree of scatter which does not improve under the action of traffic. The 95% confidence limits are of the order of ± 0.8 - 3.0 kN which are the equivalent of about 10 - 40% of the value being measured. Such variations probably derive from experimental method. They are unlikely to be caused by actual variations of the material composition of the specimen because these are more likely to be reduced under trafficking in the same way as density variations were reduced.

6.2.2 (cont.)

(iii) There is a significant correlation between the stability and air voids of a core specimen. Stability increases as the air voids value decreases in a simple exponential relationship of the form

$$S = aV^{-b} \qquad 6(4)$$

where S = Marshall core stability at 50°C (Newtons)
V = air voids (%) (i.e. a.v.u.)
a, b are constants

A linear regression analysis was performed on the log-log form of these parameters and the results for the eight different mix designs with some subdivisions by pavement thickness are shown in Appendix VI, Figures VI. 59-71. General observations from these figures are:

(a) For the same aggregate gradation, the stability becomes more sensitive to air voids as the asphalt content decreases i.e. the slope becomes more negative (Mixes A,B,C, Figures VI. 59,61,64, and Mixes D,E, Figures VI. 65,66).

(b) For Marshall - designed mixes with 80/100 pen. asphalt, the 9.5 mm aggregate has slightly higher stability than the 16 mm mix (Figures VI. 60-63, 67-69). This observation is surprising because it is generally recognised that stability tends to decrease as the maximum aggregate size decreases. However, the difference between the two mixes is very small, probably because the angular nature of the crushed aggregate has a dominant influence. Hence the stabilities of these mixes are not very sensitive to maximum aggregate size, a conclusion which becomes significant later in the consideration of layer thickness.

(c) The high viscosity Marshall-design mix, B, has a higher stability than the low viscosity Marshall-design mix, D, cf. Figures VI. 61 and 65.

(d) Pavement thickness, and consequently the internal structure of the mix, may have a slight influence. For Mix B, Figures VI. 60-63, the slope of the regression line becomes less negative as the thickness increases while the stability at design air voids (4.0%) remains nearly constant. This trend however is not well confirmed by Mix F, Figures VI. 67-69, where the curve for the intermediate thickness is

6.2.2 (cont.)

steeper than those for the other two thicknesses. It is probable however that the regression line for the thin thickness, Figure VI. 67, would be much steeper but for two stray points.

(e) The core specimens tested from the Main North Road test area, though few in number, show similar relationships for stability, cf. Figures VI. 61, 70 and VI. 65, 71.

(iv) There is a similar correlation between the flow value and air voids. The flow value tends to decrease as air voids decrease although it is not nearly as sensitive to voids as is stability. However, in three cases, the flow value tends to increase, Figures VI.61,70,71. The trend in Figure VI.61 has been swung by a stray value in the bottom right corner and in the other two figures the data sets are very small so not too much importance is attached to these.

(v) The flow value is subject to as much scatter as stability, being in the order of $\pm$ 20%. Again, this scatter is probably due to experimental rather than material variations. This is substantiated by the fact that bearing capacity values computed from both stability and flow show just as much scatter.

(vi) The magnitude of the core stability tested at $50^{\circ}C$ for the stable state of Stage 4 is generally very close to the stability of the moulded design block tested at $60^{\circ}C$. This may be seen in Figures 6.9 - 14 and comparison may also be made with the $50^{\circ}C$ stability of the moulded design block estimated from equation 5(2). Similarity between the core at $50^{\circ}C$ and the block at $60^{\circ}C$ is coincidental but it does demonstrate the very much lower measured stability of a core sample. The explanation already proffered for this is the difference in interparticle structure between the two cases, q.v. 2.3.2.

(vii) Flow value seems to be an important factor in the compaction characteristics of a mixture. Since the stabilities of all the mixes are similar and the flow values cover a wider range, the flow value seems to have a better prospect of providing an explanation for the wide range of densification characteristics. Thus an hypothesis is tentatively proposed for later consideration:

6.2.2 (cont.)

"The flow value, being a measure of the deformability of a mix, is a major factor in determining the compaction characteristics of an asphalt concrete mixture".

6.2.3 Bearing Capacity

The indicator of bearing capacity, q.v. 2.3.2, eqn. 2(10), though determined from the empirically based values of stability and flow, has at least a modicum of theoretical justification. Was it any more consistent in indicating the condition of the surface course?

(i) The bearing capacity generally increases under the action of traffic for each test condition although this was sometimes masked by scatter.

(ii) As an indicator, bearing capacity bears no better relationship to the loading conditions than does stability. It was hoped that it might produce a unique stable state value for each testing condition but the ranges obtained were wide: Stage 1, 500–1000 (kN/m^2); Stage 2, 600–1150; Stage 3, 800–1200; Stage 4, 900–1300; Stage 5, 1200–1500. One of the better examples, however, is that of Strips 16 and 18 (Figure 6.11) where the stabilities are low and quite different in the final stable state and yet the corresponding bearing capacities are in the normal range and close to one another. For a mix such as these with a stability that is nearly constant over a wide range of mix designs, bearing capacity is no better an indicator than air voids or stability. For mixtures which have stability and flow values that are more sensitive to mix design than these, the situation may be very different.

6.2.4 Squareness Functions

The squareness function test, which was applied to all the Testing Track data during the computer analysis, was intended to give some confirmation to the stable state theory. It was argued that if the mix were to reach a single stable state under certain imposed loading conditions then all areas of the pavement undergoing traffic should tend to reach a uniform level of indicator after extensive trafficking. This uniform level is the stable state value of the indicator.

6.2.4 (cont.)

The postulate may be tested by examining the calculated squareness functions tabulated in the right-hand columns of Table VI-I. The functions should be read in vertical groups of two or three determined by the stage of loading given in the second column. For example, Strip 13, during Stage 1 loading shows an increase of squareness function for air voids from 0.67 at substage 11 to 0.73 at substage 12 of trafficking. The corresponding functions for stability and bearing capacity are 0.71 to 0.71 and 0.72 to 0.73 respectively.

Study of the results reveals that the trend to a squarer function is certainly not highly significant. One of the more significant examples of a positive trend is Strip 14. The increases in air voids function of 0.69 to 0.80/0.76, 0.70 to 0.80, and 0.76 to 0.80 for Stages 2, 3 and 4 respectively do show a tendency to a greater squareness. The trends, however, are not as obvious as was hoped. The function is expected to increase slightly as the loading increases in severity because the indicators themselves are increasing. Any other trends within each stage must now be classified as insignificant. Why is this so when the graphed densification functions seem to provide such strong evidence for the stable state theory?

The explanation of the lack of significance probably lies both in the function itself and in the veracity of one of its basic assumptions. In the function itself, the basic area was calculated using the maximum indicator value as height, Figure 6.4. If one value was abnormally high and the area was average or small the squareness function would be drastically lowered. This is particularly evidenced by very low values such as 0.22 for the air voids function of Strip 11, substage 12, but it is also likely to be a contributing factor in many other cases. The other possible explanation of insignificance is the veracity of the assumption that initial conditions have no effect on the stable state conditions. If this were not true then much of the curved nature of the profile of values across the path would remain in succeeding stages. The effect of initial density is discussed later in section 6.4.

6.2.4 (cont.)

There is one further possible explanation for the lack of significance. It may be noticed from Table VI-I that the data from Test Series 2 tends to show a rather more consistent positive trend. This suggests that the non-uniformity of traffic in Series 1 may have adversely affected the squareness function. In addition to that, if horizontal stresses under the vehicle tyres were high they could cause distortion of the simple situation envisaged in developing the squareness function postulate.

6.2.5 Transverse Surface Profiles

The recording of the transverse surface profiles by a simple profilometer was an imprecise measurement because its sole intention had been to detect the onset of excessive deformation and distortion. In fact, as an indicator of pavement performance, it yielded more than this. The results are depicted in Figures 6.17, 18 and are presented in detail in Appendix VI, Table VI-II and Figures VI. 39-58.

(i) The mean permanent deformation increased under the action of traffic. The magnitude of this was fairly consistent between strips in Test Series 1 giving a final mean deformation of approximately 2.2 mm, that is approximately 4.5% of the surface course thickness. In Series 2 the magnitude was not so consistent varying from 1.4 to 4.5 mm or approximately 2 - 12% of the surface course thickness.

(ii) On the assumption of a constant mass of material per unit area, the mean deformation/unit thickness should correspond with the increase in density. In Series 1, the approximate mean deformation is 4.5% compared with density increases in the range of 2 - 4%. Allowing for some lateral material movement which did occur in Series 1, see Clause (vi), and also for the inaccuracy of the profile measurement, this is a satisfactory correlation. In Series 2, there is a similar correlation with the measured deformation tending to exceed the corresponding increase in density by approximately 50%. In the best case, Strip 25, the two are nearly equal at 2.3% and 2.2% respectively and in the worst case, Strip 28, the former is double the latter with 12.5% and 6.2% respectively. Lateral movement was not as significant here as in Series 1 and so the difference is probably attributable solely to experimental error.

6.2.5 (cont.)

(iii) Increases in mean permanent deformation were generally immeasurable or small in Stages 1 and 3, the low temperature conditions, and they were significant in Stages 2, 4 and 5, the higher temperature conditions. However, there were a number of exceptions to the former generalisation for Stage 3. Strips 14, 15, 16, 17 and 18 showed increases in deformation that were significant. Since these were accompanied by a negligible change in density it is considered that the difference is probably the result of experimental error. Another possible cause is thermal expansion because the profiles for Stage 2 were measured while the pavement was still hot. However, the contribution of thermal expansion is small.

(iv) The larger increases in mean deformation are generally accompanied by an increase in distortion. A measure of distortion is given by the average deviation from the mean depicted in Figure 6.18. The deviation is generally high in Stages 2 and 4 and sometimes very high in Stage 5. It generally decreases in Stage 3 when the combination of heavier load and low temperature restore some uniformity to the deformations.

(v) The uneven distribution of traffic in Series 0 and 1 before the testing vehicle was modified resulted in more surface distortion and higher average deviations for those series than for Series 2. A study of the exaggerated profiles included in Appendix Figures VI. 39-48 shows predominant wheel ruts at both edges of the path. In Series 2, however, the deformation is much more uniform across the path until the severe conditions of Stage 5 when distortion becomes significant.

(vi) The postulated mechanism of rutting is given graphic support by the more severely distorted pavements, e.g. Strips 02, 04, 12, 16. The mean permanent deformations in the trafficked path are close to those of the other strips and compatible with the change in density so that the volume of material involved is approximately constant. The ridges must therefore have formed from material squeezed out of the valleys and thrust upwards adjacent to the loaded area. In extreme cases, where ridges have developed immediately

6.2.5 (cont.)

outside the trafficked path, it seems likely that some material has been thrust out from the area of the trafficked path. This would cause a net loss of material mass from the trafficked path and would increase the apparent mean deformation.

6.2.6 Surface Texture

Changes in the surface texture of the pavement due to traffic action may have a significant effect on tyre-pavement friction or skid resistance. One of the major reasons for the upper limit on design density, namely that giving 2-3% air voids, is that the pavement will always maintain an adequate margin from the zero air voids condition. As the voids approach the Z.A.V. condition, flushing of the binder to the surface is probable with a consequent loss of tyre-pavement friction. Only the significant examples of texture from the photographic record are included in this thesis.

(i) In the first preliminary test strips, there was a noticeable change in surface texture over the period of high loading and very high temperature conditions, 620 kN/m^2 and 60^oC at middepth. There was an erosion of fines from the surface exposing some of the larger aggregate particles, as has already been seen in Figure 5.6(a),(c). There was also on Strip 02 a marked incidence of microcracking, Figure 5.6(d). This appeared to be a prematurely induced ageing effect with a breakdown of the asphalt film between clumps of particles.

(ii) Both the low viscosity mix on the Main North Road and its replica on the Testing Track, Strip 16, showed fine slicks of free bitumen on the surface after 21 months and Stage 2 of trafficking respectively. They were then at air voids contents of 2.3% and 1.3% respectively. This is discussed further in 6.7. An important point however is that the texture on the Main North Road strip was much smoother and closer than on the Test Track strip. This appeared to be the direct result of the presence of engine oil, tyre rubber and fine dust on the North Road strip, and it highlights one of the artificial aspects of the Testing Track.

6.2.6 (cont.)

(iii) In the main track test series the changes in texture were slight, cf. 6.6.1 and Figure 6.21. Lean mixes tended to retain their dry appearance even with some tyre black, although the texture did close very slightly. Normal mixes generally retained a good sandpaper-type texture throughout testing. Richer and finer mixes generally gave a closer texture but never became very smooth. Even Strip 16, mentioned in the previous clause, retained a large degree of sandpaper-type texture.

6.2.7 Skid Resistance

A study of skid resistance was made to determine whether this changed significantly for the changes occurring in surface texture. The data determined using a pendulum skid tester are given in Table 6-I.

(i) The values are high, all being in the region of 62-75 for critical wet conditions and implying a minimum coefficient of friction of approximately 0.62-0.75.

(ii) The values for each strip are close to one another implying that asphalt type and viscosity for this constant aggregate gradation has little influence on this measure of skid resistance. This covered a range of coarse textures, such as Strips 13 and 18, to fine close textures, such as Strips 14 and 16. The value remains high for Strip 16 despite the appearance of some bitumen slick on the surface.

(iii) There is negligible change in skid resistance during trafficking, the mean value varying by no more than ± 2 for any strip.

(iv) While these pendulum-tested skid resistance values are high, the fine, close-textured surfaces are likely to induce the onset of hydroplaning sooner than the coarse-textured surfaces. The presence of engine oil, tyre black and fine dust on normal highway pavements is likely to be a major factor in this.

6.3 THE INFLUENCE OF LOADING CONDITIONS

6.3.1 Traffic Volume

In nearly every densification function from the Testing Track there is a rapid change in the indicator in the first

TABLE 6-I SKID RESISTANCE RECORD
TEST TRACK, SERIES 1

STRIP NO.	LOCATION	Skid Resistance* at end of Testing Stage Number 0	0	1	1	2	3	4
		dry	wet	dry	wet	wet	wet	wet
11	A	100	70	100	66	67	69	70
	B	101	65	100	65	65	66	65
	C	100	68	100	67	65	66	66
	D	99	70	99	68	68	69	68
12	A	96	69	101	70	69	69	70
	B	98	65	100	66	65	68	67
	C	98	65	101	69	65	66	-**
	D	95	65	101	69	67	66	69
13	A	88	70	95	69	75	70	76
	B	98	70	99	65	65	67	68
	C	99	68	105	71	66	67	70
	D	103	70	104	70	69	70	75
14	A	92	64	102	66	69	64	67
	B	98	60	100	64	65	65	68
	C	100	65	103	65	66	65	68
	D	100	68	100	65	69	68	69
15	A	96	66	100	70	75	72	74
	B	100	70	101	70	70	68	70
	C	98	70	100	70	70	70	74
	D	95	69	101	70	73	71	75
16	A	91	61	98	64	67	70	70
	B	98	62	100	62	65	68	67
	C	92	61	100	65	67	68	66
	D	100	72	98	65	68	70	71
17	A	93	65	100	69	70	69	69
	B	98	68	100	65	66	67	68
	C	100	70	100	69	67	69	68
	D	95	66	99	67	68	70	70
18	A	100	65	103	69	69	67	70
	B	96	65	100	65	66	69	70
	C	100	67	100	68	66	68	69
	D	95	64	100	65	69	70	73

*All values measured by pendulum skid tester.
**No reading because of slope from excessive surface distortion.
N.B. Each value is average of 5 readings. Locations A,B,C and D refer to the four quarters of the path width at the centre of each strip.

6.3.1 (cont.)

500-1000 passes of the testing vehicles. This is shown as a steep initial gradient on the curves in Figures 6.7 to 6.14, negative for air voids and positive for stability and bearing capacity. Similarly, on the Main North Road and Northern Motorway test areas there is a rapid change during the first 100,000-200,000 passes of mixed vehicular traffic, Figures 6.15, 16. It is fair to state then that, in most cases, under repeated vehicle loading, the initial change in indicator is rapid. The only cases where this does not hold are those that are in, or very nearly in, the stable state for the new loading conditions. In these cases the curves have a zero or very small gradient.

The magnitude of this initial rapid change and the continuing rate of change seem dependent on the relative magnitudes of the initial and stable conditions. For example, Strip 11, Mix B, in Figure 6.9 undergoes negligible densification to Stage 1. In Stage 2, however, with the higher temperature, it densifies from 4.0% to 2.6% air voids during 10,000 passes of the testing vehicle and one half of that density gain is achieved in the first 1000 passes.

The continuing rate of change seems to vary in nature. Where a large change in state is occurring, the rate of change remains fairly high, decreasing slowly, e.g. Strip 13 Stage 2; Strip 27 Stage 2; Strip 16 Stage 2; Strip 26 Stage 2 and Strip 28 Stage 4. Where a smaller change in state, of the order of 1%, is occurring the rate decreases rapidly to a low value becoming zero after a few thousand passes, e.g. Stage 3 for most strips, Strip 1 Stage 1, Strip 22 and 25 Stage 1, etc.

The scatter of indicator values generally decreases during the period of trafficking, cf. 6.7.3.

6.3.2 Load and Temperature

The relative influences of imposed load and material temperature may be assessed by comparing the effects of each combination of testing conditions from Figures 6.7-14.

6.3.2 (cont.)

(i) The light conditions of Stage 1, low load and low temperature, have a significant effect in compacting the pavement. While the effect may not be as large as in some other stages, it is sufficiently large to increase the indicator by 1 to 2% in most cases. The conditions caused little change in the optimum and lean mixes, Strips 11, 13, 15, 17, etc., some of which had high air void contents. However they caused a significant change for the rich mix, Strips 12 and 14, even though it was low in air voids content.

(ii) In all cases, the higher temperature of Stage 2 testing conditions caused a distinct change in indicator. This change was usually of the order of 1.5% although it was rather less for the lean mixes C and E on Strips 15, 17 and 18, and for the mixes of low air void content such as the rich Mix A on Strip 12. The calculated decrease in binder viscosity that produced these changes was from 95 to 2.9 m^2/s for the 80/100 pen. asphalt and from 11.9 to 0.61 m^2/s for the 180/200 pen. asphalt, the effect on stability may seem less than on air voids from the figures but this is a scale effect and it is, in fact, comparable to the effect on density.

(iii) Stage 3 testing conditions with high load and low temperature generally have a negligible effect on the stable state achieved during Stage 2. Exceptions to this are Strip 18 where the density and stability increased by approximately 1%, and Strips 23 and 25 where stability increased with negligible change in density. Strip 22 is a special case because it did not undergo Stage 2 due to failure of the heating elements. The evidence of Strips 23 and 25 is not considered significant in the light of the higher accuracy of the density measurement over the stability measurement, and of the strong relationship between stability and density discussed earlier. The evidence of Strip 18, however, is more significant and may be a lead to understanding the mechanism of compaction. For instance, in the case of a lean mix where the binder film is thin it is possible that a 20-fold increase in viscosity is insufficient to counteract a two-fold increase in load, which is the situation for Strip 18. This is not corroborated though by Mix C, also lean, on Strips 15 and 17, although there the viscosity increase was 35-fold. This matter is also discussed in 6.6.2. Despite the fact that the

6.3.2 (cont.)

conditions of Stage 3 are less than or equal in severity to those of Stage 2, they are still sufficient to cause a significant decrease in the scatter of indicator values. This implies that Stage 3 conditions are very close in severity to those of Stage 2, close enough to increase uniformity but not great enough to change the stable state conditions.

(iv) The high load, high temperature and low viscosities of Stage 4 generally induce a further significant change in indicators. However, the change is smaller than in Stage 2 and generally of the order of 0.5%. The mixes of low air void content undergo negligible change, and so do the lean mixes on Strips 15, 17 and 18. Where there was a differential in construction density, the initially denser mix has only a very small change while the less dense mix has a greater change and approaches close to the denser one, e.g. Strips 15 and 17, 11 and 13.

(v) In Test Series 2, a fifth stage of abnormally high temperature, $50^{o}C$, was run to provide an extreme limit of testing conditions. All strips have a further significant change in indicator and plant values are reached in every case. This makes it very clear that any stable state the pavement reaches is stable only for the severest loading conditions that have existed for a reasonable period. The stable state can be changed if the pavement is subjected to more severe loading conditions.

6.3.3 Magnitude of Stable State Conditions

The magnitude of the stable state conditions reached in each case equals or exceeds predictions, yet the magnitude of imposed load and of material temperatures are representative of normal highway conditions. Full comparison of the track and highway studies is made in section 6.7 but if Figures 6.9 and 6.15(a), and 6.11 and 6.15(b) are compared it will be noticed that there is not much difference between the stable states achieved after Stage 2 on the Track and on the Main North Road. The stable states for Stage 4 on the Testing Track exceed the normal conditions slightly and the stable states for Stage 5 are clearly higher still. Hence there is evidence to infer that reduced vehicle speed, or increased

6.3.3 (cont.)

load duration is producing slightly higher densities as well as its expected effect of accelerating the densifying process, cf. 6.7.3.

6.3.4 Some Thoughts on the Mechanism of Influence

The observations in this section on the influence of traffic volume and load and of temperature prompt some thought on the mechanism of compaction by these factors. Imposed load stresses tend to force the aggregate particles closer together in an intimate matrix. This is resisted partly by interparticle bearing and partly by the viscous resistance of the binder. At low temperatures, high viscous resistance is likely to inhibit any alteration of the general interparticle structure by imposed stresses although it will probably allow particles to move closer together without significant change in disposition. This explains why Strip 11 with a stable dense structure was unaltered in Stage 1 of testing. At higher temperatures, the viscous resistance is lower permitting more flow of the binder and hence some reshuffling of particles into a denser, stronger structure. This may be accompanied by reorientation of some particles.

Essentially, it seems that a certain amount of work energy is required to cause restructuring as well as densifying of the mixture. This energy level is more effectively reached by lowering the viscous resistance than by increasing the imposed load.

6.4 THE EFFECT OF CONSTRUCTION DENSITY

6.4.1 Density

The effect of initial conditions on the magnitude of the stable state conditions determines how critical are construction specifications. Mixes A, B, C and D were tested for this at the Testing Track, Figures 6.9, 12, and what was intended to be Mix B but designated Mix J, was tested on the Northern Motorway Test Area, Figure 6.16.

The results are presented in Figures 6.19, 20 in a special form that highlights any dependence on construction conditions. The density at a certain stable state condition is plotted against

6.4.1 (cont.)

the construction density appearing as one point on the graph. These densities are plotted as percentages of design density so that there is a ready comparison between different mixtures. All such points for those testing conditions but different construction conditions are joined by a line. If the stable state density is independent of construction density then these points will lie on a horizontal line indicating that no matter what the construction density this stable state density will be reached under these loading conditions. A line of zero slope is then indicative of "zero significance". In contrast, if the stable state density is highly dependent on construction conditions then the line will have a positive slope as illustrated.

The data from the Testing Track is presented in this manner in Figure 6.19. For each mix except Mix A there is a tendency for a high significance under Stage 1 testing conditions, gradually decreasing to only a slight significance at Stage 4. This seems to indicate that the less dense and strongly-packed structure associated with lower construction densities is not forced into the form of the highly-packed structure until heavy loads and high temperatures are imposed.

The data for Mix A in Figure 6.19(a) follow the same general trend although the Stage 1 curve is reversed in slope. This anomaly is the result of the heating failure on that Strip during Stage 2.

The data from the highway test areas are presented in the same manner in Figure 6.20. Both mixes on the North Road area show a fairly strong dependence on initial conditions. The mix on the Motorway area shows a fair but slightly lesser dependence on initial conditions and this is independent of base flexibility. This lesser dependence probably stems from the more open grading of Mix J which has a higher flow value and a looser particle structure that is easier to reorientate.

6.4.2 Permanent Deformation

Construction density surprisingly has little apparent influence on either the magnitude or the variation of permanent deformations. Reference to Figures 6.17, 18 shows that the normal

6.4.2 (cont.)

mix on Strip 11 performs slightly better than the initially less dense mix on Strip 13, the lean mix on Strip 15 performs no better than on Strip 17, and the rich mix on Strip 12 performs no better than on Strip 14. This is surprising because the density change is greater on the less dense strips than on the dense ones so that the deformation is expected to be larger for the less dense strips. Obviously, if this did happen, the measuring technique was insensitive to the difference.

6.5 THE EFFECT OF SURFACE COURSE THICKNESS

6.5.1 Density

A thin surface course is expected to have a greater stability under imposed load due to the interaction of confining frictional stresses at the boundaries. If this were significant it ought to be evidenced by a higher resistance to compaction than for thicker courses of the same mix.

In Series 2, Strips 21 and 27 (38 mm thick), 22 and 25 (60 mm) and 23 (95 mm) represent a range of surface course thickness approximately equivalent to 2, 4 and 6 times the maximum aggregate size for Mix B. Reference to Figure 6.12 shows that it is difficult to distinguish a significant effect of thickness from the effect of initial density. However, on the confirmed basis that effects of initial density become very slight under Stage 4 or 5 testing conditions, there does seem to be a slight strengthening effect in the thinner layers. The 38 mm strips have a density approximately 0.5% less than the 60 mm strips which are in turn approximately 0.8% less in density than the 95 mm strip. Clearly the effect is small.

The finer mix, Mix F, on Strips 24 (65 mm thick), 26 (43 mm) and 28 (21 mm) demonstrates a somewhat lesser influence of thickness on density, the thinner strips both densify rapidly so that after Stage 4 there is only 0.8% between all three. Although it is surprising that the 65 mm strip did not densify further in Stage 4, it is unreasonable to attach much significance to these differences.

6.5.2 Permanent Deformations

Reference to Figure 6.17(c), (d) shows that the thinner strips incur less permanent deformation than the thicker ones, especially when the magnitude of equivalent density change is considered. However, on a deformation/unit thickness basis there is very little difference.

The distortion or the incidence of rutting within the wheelpath is profoundly affected by the thickness, Figure 6.18(c), (d). In all cases the thinner strips suffer less distortion than the thicker ones. Hence distortion appears to be the factor most affected by surface course thickness.

6.5.3 Variation of Indicator with Depth

The variation of indicator with depth may be observed in Figure 6.14 which illustrates the characteristics of the two halves of the cores from the 95 mm thick Strip 23. The mix is Mix B, a normally designed 16 mm mix with 80/100 pen. asphalt. Each core, after being measured for density and thickness, was cut in to two pieces approximately 48 and 43 mm thick, with a 4 mm width of cut. These cut portions were then submitted to all the standard core measurements.

The upper half of the pavement represented by curve 23A has a considerably greater density, stability and bearing capacity than the lower half, curve 23B. Over Stages 1 and 2 this is clearly more than a preservation of the initial conditions because the curves are diverging. This is the expected result because the temperatures are usually higher and the vertical stresses higher in the upper region. Thus, in reality, the loading and environmental conditions are generally more severe in the upper than in the lower region.

During Stages 3 and 4 this trend is still apparent in the stability and bearing capacity functions, but it rapidly becomes insignificant in the density function. Inspection of the testing log and temperature chart for that period indicates that this anomalous trend is the result of experimental technique. The time of testing was in autumn and more artificial heating was being

6.5.3 (cont.)

required to achieve the standard temperatures, especially in this very thick strip. Thus the temperature became fairly uniform with temperatures in the lower regions occasionally being higher than in the upper regions. This implies, inter alia, that density differentials are primarily a result of temperature differentials and that the effect of "load-spreading" is secondary. This was certainly the implication of the elastic analysis described earlier which demonstrated only a 3 to 5% load-spreading effect at the base of the 50 mm thick surface course considered. Since viscous behaviour will tend to relax such a load-spreading effect, the total expected influence of load-spreading is only of the order of 2 to 3% reduction of the imposed load stress. With these considerations, the data of Figure 6.14 corroborates those suppositions.

6.6 THE INFLUENCE OF MIX DESIGN

Variations in mix design that were tested did not include either aggregate gradation or aggregate shape. These two parameters were held constant. The aggregate was crushed in all fractions although the Testing Track mixes contained some natural material in the fraction finer than 0.3 mm. The gradation was dense as illustrated earlier in Figure 5.3.

6.6.1 Asphalt Content

A mix is generally designed by adjusting both the asphalt content and the aggregate gradation so that the design air voids, stability and flow values fall within a specification. Asphalt contents in these tests were deliberately designed to provide a range of rich, normal and lean mixes for the constant gradation (i.e. excess, optimum and deficient binder contents). Mixes A, B and C are rich, normal and lean designs respectively of the aggregate bound with 80/100 pen. asphalt. Mixes D and E are normal and lean designs respectively of the same aggregate but bound with the softer 180/200 pen. asphalt.

(a) High Viscosity Mix

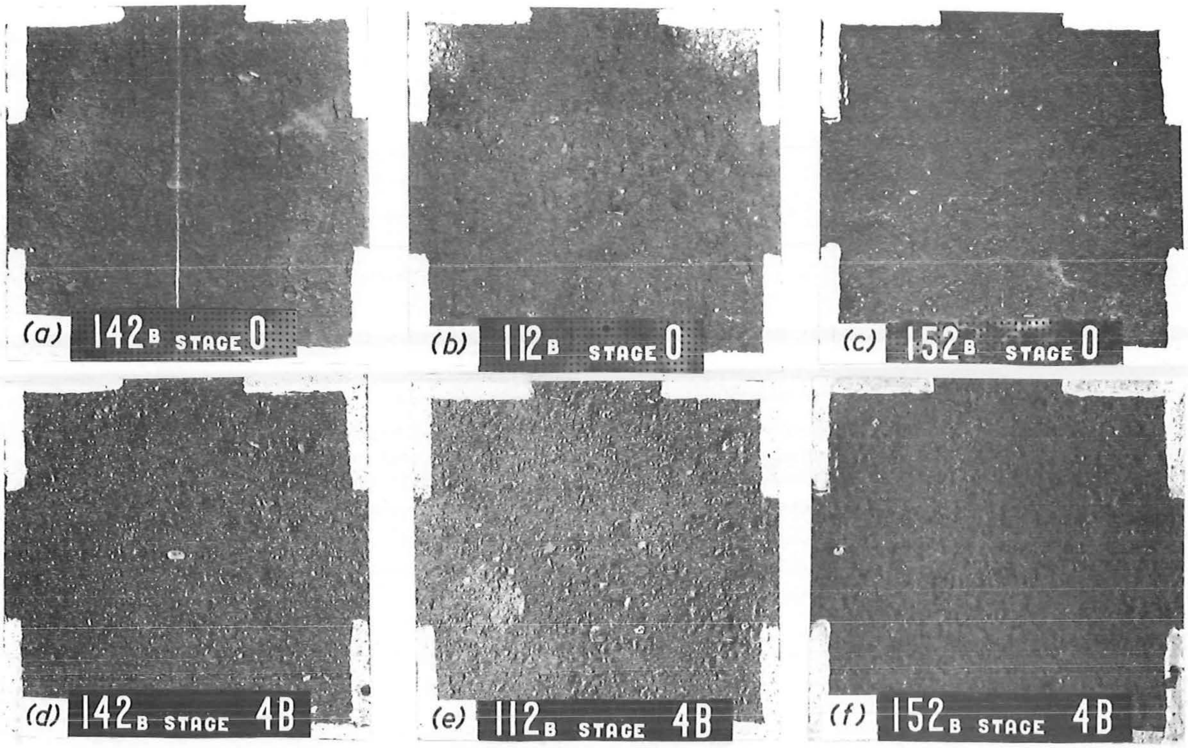
As produced, the high viscosity mix had air voids contents

6.6.1 (cont.)

of 1.9, 2.5 and 5.2% for rich, normal and lean designs respectively. This differential in density is preserved in the behaviour under traffic on the Testing Track, Figure 6.10(a). Note that the Figure summarises the data in Figure 6.9 and that the curves represent the lower constructed density type of each pair of each mix. The predicted density in each case is nearly reached by the end of Stage 2, although the rich mix A exceeded predicted density from near the beginning of Stage 1. Core stabilities, Figure 6.10(b) are all closely bunched with much less differential than at design. They all consistently fall well below design stabilities even with the reduction of test temperature from 60°C to 50°C. In this instance, the core stabilities are effectively about 40–50% of the design stabilities on the basis of an equivalent 60°C test temperature. The bearing capacity, ib. (c), of the rich mix is consistently less than those of the normal and lean mixes by about 10% but again there is much less differential between the mixes for the pavement cores than for the design values included on the graph.

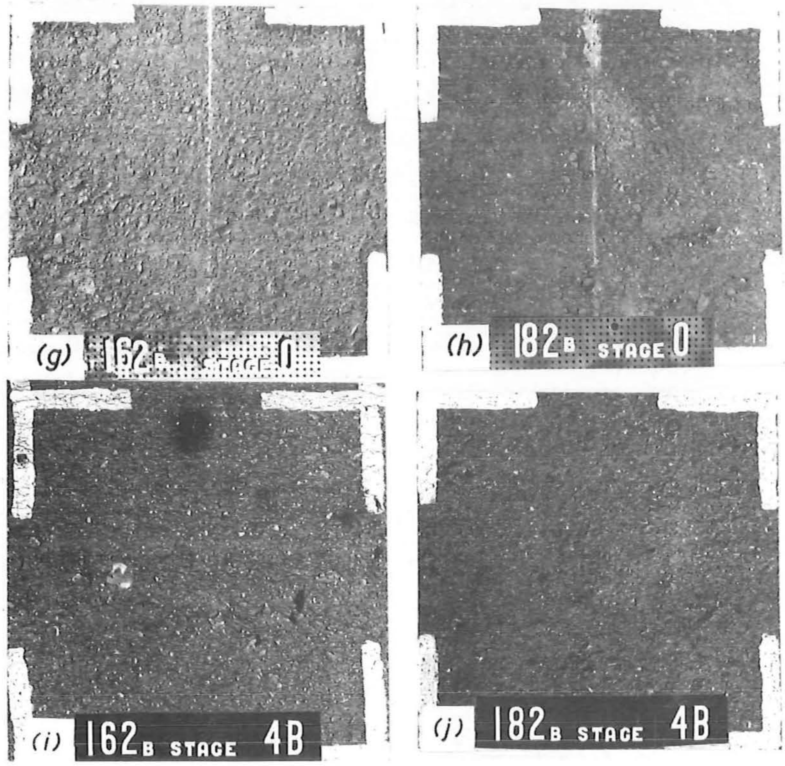
The permanent deformation characteristics of these mixes also follow expected trends. The rich mix on Strips 12 and 14 tends to have a slightly larger mean deformation, Figure 6.17(a), and much greater degree of distortion and rutting, Figure 6.18(a), than the normal mix on Strips 11 and 13. Both the normal mix on Strips 11 and 13 and the lean mix on Strips 15 and 7 are very stable in respect of permanent deformation. The lean mix tends to have only slightly less mean deformation and distortion than the normal mix.

The effect of asphalt content on surface texture and surface deformations is shown in the photographs of Figure 6.21. Photographs (a), (b) and (c) are representative views of texture after construction but before trafficking for the rich, normal and lean mixes respectively. Photographs (d), (e) and (f) show the texture for the same mixes after Stage 4 of testing. The texture of the rich mix is very close and it shows signs of distress from the excessive deformation and rutting similar to that illustrated in (k). The normal mix has closed in texture but retains a well-textured surface. The texture of the lean mix has changed little. Both the normal and lean mixes showed some erosion.



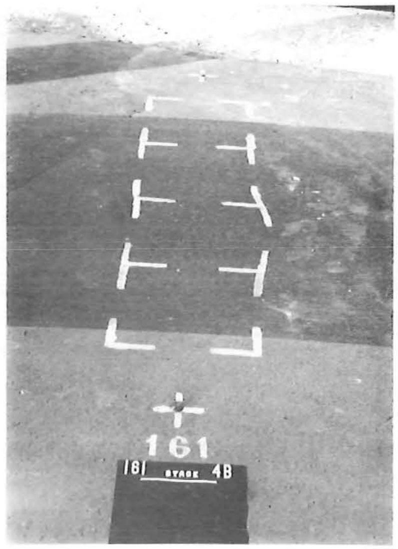
Rich, 6.8% A.C. Optimum, 6.1% A.C. Lean, 5.2% A.C.

High Viscosity Asphalt - Before (top) and After (btm) Trafficking



Optimum+, 6.1% A.C. Lean, 5.2% A.C.

Low Viscosity Asphalt - Similar above



(k) Horizontal Deformation of Low Viscosity Optimum+ Mix

Figure 6.21 DEPENDENCE OF SURFACE TEXTURE ON BINDER CONTENT UNDER TRAFFIC ACTION - TESTING TRACK

6.6.1 (cont.)

(b) The Low Viscosity Mix

Similar trends are evident for the low viscosity (180/200 pen. asphalt) mix. Only one strip (16) of the normally designed mix, Mix D, and one strip (18) of the lean mix, Mix E, were tested in Series 1. In addition, Strips 01 and 07 in the preliminary series were both of the normal design although the testing sequence was not the same as in Series 1. The normal and lean mixes have design air void contents of 3.5 and 5.6% respectively.

Reference to Figure 6.11, which gives the densification functions for the normal and lean mixes, shows that the differential in design density is replicated as a differential throughout trafficking in each testing condition. The lean mix reaches its predicted voids during Stage 3 but the normal mix exceeds its predicted density from Stage 2 onwards. Stability and bearing capacity both show marked differentials throughout in contrast to the negligible difference in design values for both mixes.

In resistance to deformation, the lean mix here is far superior to the normal mix, Figures 6.17(b) and 18(b). The difference is most noticeable in Stage 4 when the normal mix has deformed an average of 3.4 mm and is badly distorted with an average deviation of 2.6 mm and a maximum rut depth of 8.3 mm. The lean mix in contrast has an average deformation of 2.5 mm, is virtually undistorted with a deviation of 0.3 mm and the maximum rut depth is only 2.9 mm.

In surface texture, Figure 6.21, the lean mix has remained fairly dry looking and coarse, ib. (j), while the normal mix has a close, fatigued-looking texture with a high incidence of bitumen slick in small filaments on the surface, ib. (i). This is not surprising because the normal mix has reached 1% air voids at that stage. The behaviour of the normal mix is discussed in more detail in section 6.7, but it appears from these observations to be a rich rather than a normally designed mix.

For both binder viscosities, it is evident that differences in design density due to asphalt content are replicated in the pavement. This is definitely not true of stability and bearing capacity

6.6.1 (cont.)

values which fall well below predicted levels and have little relation in differential to those levels for the various asphalt contents. Asphalt content however has its greatest effect on the deformability of the pavement. Rich mixes deform a great deal and suffer severe distortion, normal mixes deform within acceptable limits without much distortion, and lean mixes deform acceptably with negligible distortion.

6.6.2 Asphalt Viscosity

The effect of asphalt viscosity on compaction can be deduced from two sources: (i) from two mixes having the same basic mix design but binders of different viscosities, e.g. Mixes B and D which both have 6.1% binder content but of 80/100 pen. and 180/200 pen. asphalt respectively, and (ii) from different testing temperatures for both binders. For conditions of a constant imposed load, therefore, the combination of two binder types and two testing temperatures effectively gives four data points. Furthermore, since there are two magnitudes of imposed load, there will be two sets of four points.

For these mixes and the testing temperatures of 25°C and 40°C used at the Testing Track, the stable state conditions for the two magnitudes of load are shown in Figure 6.22. Air voids are chosen as the indicator because it is a simple common denominator and binder viscosity is plotted on a logarithmic scale. The points plotted are the corresponding stable state conditions derived from Figures 6.10 and 6.11. The influence of initial density caused some difficulty in collating this data but compensation has been made where possible. Also for this particular figure, the state achieved in Stage 3 has been regarded as the stable state for those conditions. The data exhibit a fair degree of scatter especially for the larger load, but nevertheless the basic trend is clear for both magnitudes of load. Lower binder viscosities produce lower air voids or higher densities in a definite semi-logarithmic relationship. The figure also shows that the magnitude of load has a varying relative effect dependent on viscosity. These conclusions lend stronger substance to the discussion on load and

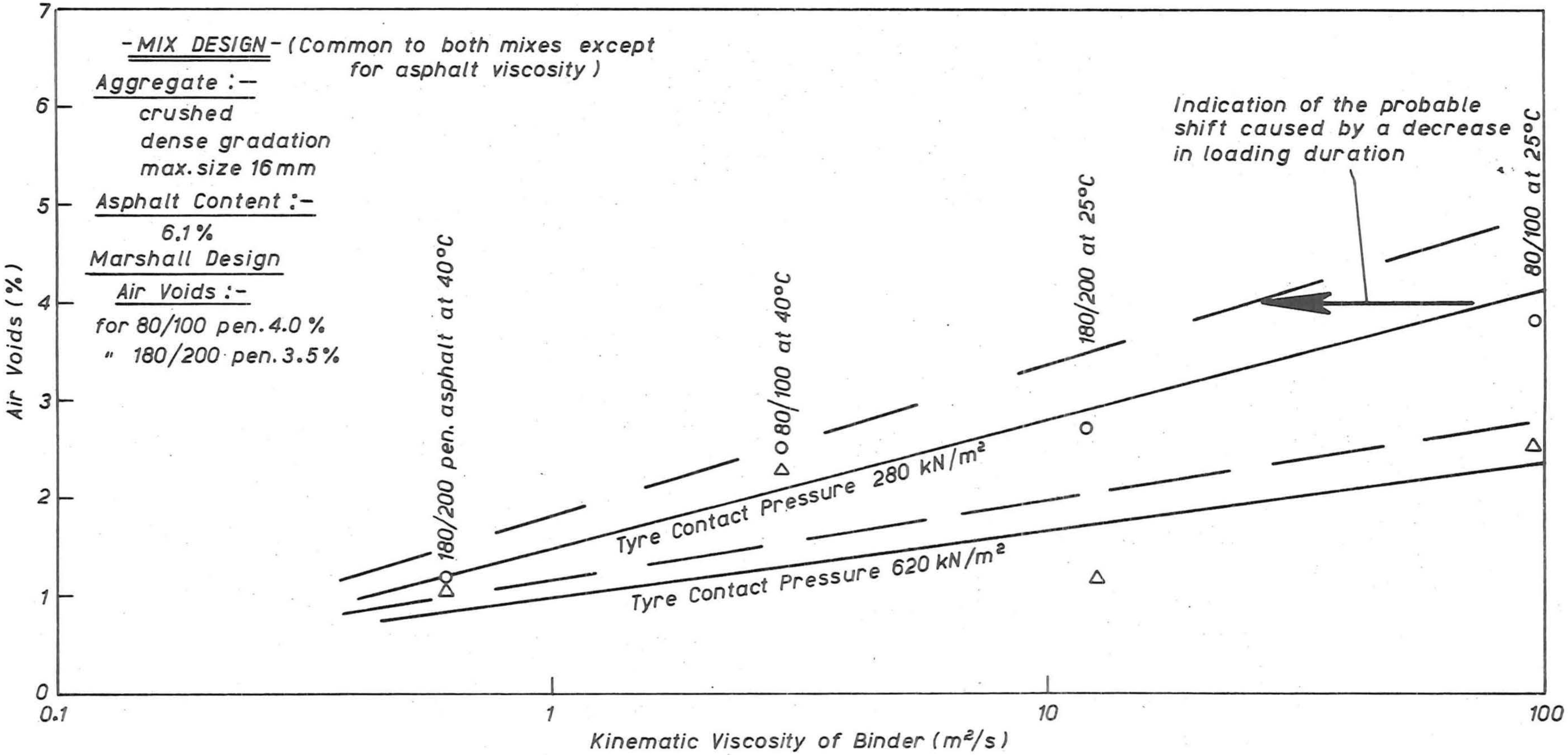


Fig. 6.22 THE EFFECT OF ASPHALT VISCOSITY ON STABLE STATE DENSITY FROM TEST TRACK DATA

6.6.2 (cont.)

temperature in section 6.3.2.

The stable state air voids contents shown in the figure are less than predicted voids for nearly all the range of loading conditions considered. One possible reason for this is the slower speed, and hence longer loading duration, of the testing vehicle. Application of the linear viscoelastic theory has indicated that the principle of time-temperature superposition is usually valid for asphalt concrete mixtures. This principle will help to indicate the effect of increased speed, or reduced loading duration, in terms of a temperature or viscosity shift. A decrease in load duration would have similar effect to an increase in viscosity. Shifting the curves to the left as shown by the broken lines in Figure 6.22 would compensate for such decrease in loading duration. This has the effect of producing a higher stable state air voids content for any given asphalt viscosity. Thus the supposition on the effect of slower vehicle speed is confirmed. The shift cannot be quantified, however, because the viscoelastic functions of the mixture have not been established and because the indicator, air voids, is unsuitable for direct interpretation by viscoelastic theory.

So far as mix design principles and the inclusion of climatic factors are concerned, such a basic relationship is clearly valuable.

6.6.3 Maximum Aggregate Size

The effect of maximum aggregate size on the behaviour of certain thicknesses of surface course may be gauged from the data from Test Series 2. Mix B is a normally designed mix with a 16 mm maximum aggregate size. Mix F is basically the same mix with the 16-9.5 mm aggregate fraction removed. It too is normally designed so that it has a higher asphalt content, 6.75%, than Mix B, 6.10%. These two mixes were laid in various thicknesses and the effect of that was discussed in section 6.5.

Comparison of the behaviour of the two mixes should be made at the same thickness, for example, Strips 25 and 24 at approximately 60 mm thickness. Comparison of these strips in Figures 6.12 and

6.6.3 (cont.)

6.13 shows little of significance except that the finer mix on Strip 24 does not reach predicted density until the final stage. However, Figures 6.17(c), (d) and 6.18(c), (d) show that the finer mix sustains twice the mean deformation of the coarser mix (2.9 and 1.4 mm respectively) and it also suffers much more distortion and rutting (a deviation of 2.0 mm compared with 0.7 mm). At the thinner thickness of 38 mm for Strips 21/27 and 26 respectively there is much less disparity in behaviour with the finer mix behaving quite acceptably. The very thin pavement of the fine mix, Strip 28, has no counterpart in the coarser mix but it behaves in a very stable manner.

The effect of maximum aggregate size, then, seems to be very closely tied to the surface course thickness. A finer mix will perform acceptably in thinner courses. In general terms, a normal mix will behave acceptably in surface course thicknesses in the range of 2 to 5 times the maximum aggregate size. The condition on 'normal' here is that the mix could be made to behave satisfactorily at greater thicknesses if the deformability or flow value were reduced, i.e. by either increasing the binder viscosity or by adjusting the aggregate gradation.

6.6.4 The Role of Aggregate Shape and Gradation

The effects of aggregate shape and gradation which were held constant throughout these tests may be gauged from their effects on the strength and deformability of the mixture. Since the flow value seems to have played a more important role than stability in the densification functions studied, deformability is probably the interesting characteristic. This is especially so because the stabilities are high and far from marginal and the mixes had thus been expected to be resistant to much further compaction.

Aggregates that produce a high flow mix are likely to cause the mix to compact more rapidly to its stable state than would a low-flow mix. Such a generalisation however might include some intractable mixes of open gradation and so a condition must be placed on 'workability'. The unintentionally gap-graded mix laid on the Northern Motorway Test Area had low workability during paving

6.6.4 (cont.)

but it is densifying as rapidly, if not more, as the normally-graded mix on the Main North Road Test Area.

There will probably be less difference in the rate of densification if rounded instead of angular aggregate is used because this seems to have only a slight effect on flow value. The stability however will be much less and this may well increase the possibility and the rate of excessive deformation and distortion.

6.7 CORRELATION BETWEEN TRACK AND HIGHWAY

6.7.1 Performance of Main North Road Test Area

The test area was on a major arterial route in a built-up area. It had a medium traffic density of approximately 8000 v.p.d. spread over two lanes and containing an estimated proportion of 10% commercial vehicles. The distribution of traffic between the lanes was not known, but the author from his observations, estimated it to be 60%/40% in the inner and outer lanes respectively. This is a high proportion in the overtaking lane but because it is in a built-up area with moderately high traffic density and with lane-changing due to the proximity of intersections this estimate is realistic. The densification functions were summarised in Figure 6.15 on this basis after the estimate used for compiling the original data as in Appendix Figure VI. 72 (80%/20%) was shown to be improbable by the trend of the data.

Figure 6.15 shows that most of the densification occurred during the first million vehicle passes/lane, that is in approximately eight months. After a further million vehicle passes/lane, the rate of densification has decreased to near zero so that the pavements seem to have reached a stable state by the 21 month period. At that stage, both the higher and the low viscosity mixes have exceeded their predicted densities by between 0 and 1%. The actual magnitude of these densities is doubtful because of the variability in the mix and consequent lack of reliable data on mix composition for an accurate calculation of zero air voids density. The variation on this job was of the order of $\pm$ 1% or $\pm$ 1 a.v.u.

6.7.1 (cont.)

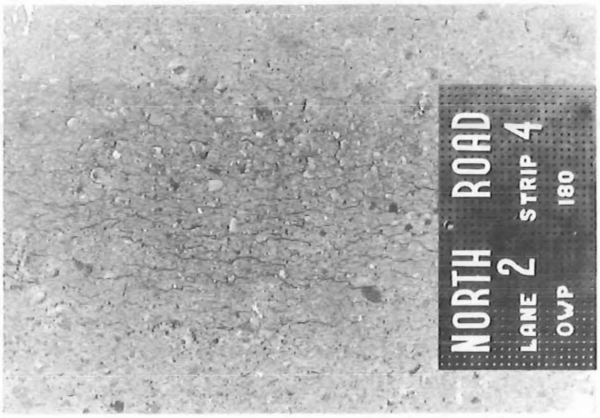
Between the wheelpaths the pavement densified little over the period of 21 months, but nevertheless the low viscosity mix exceeded the predicted density slightly.

After 8 months, in some regions of the wheelpaths, the low viscosity mix was exhibiting a fair incidence of asphalt slick on the surface in the form of fine filaments. This is illustrated in the photograph, Figure 6.23(a). It appeared to be the onset of flushing, although the air voids content, 3.0%, indicated that flushing was possible but unlikely. Considering that the air voids content generally increases with depth in the surface course, it seems probable that the air void content on the surface was near zero if the average air voids of the 45 mm deep cores was 3.0%. The filaments of slick were aligned longitudinally as if they had appeared in longitudinal cracks caused by transverse flexure of the pavement in the wheelpath region. The higher viscosity mix showed none of these signs at this stage, Figure 6.23(b), and neither mix exhibited significant surface deformations.

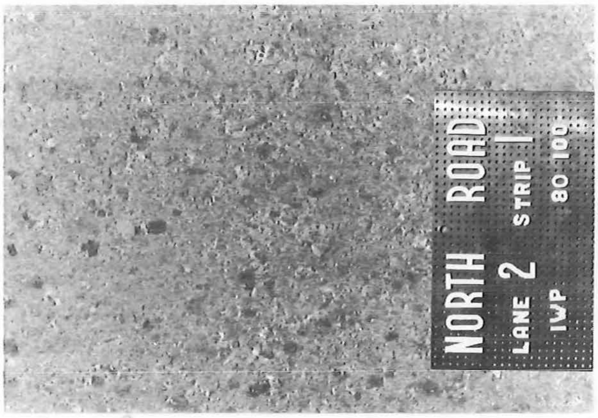
After 21 months of traffic, filaments of asphalt slick were apparent with a high incidence over all the low viscosity mix strips, in both lanes and, to some extent, between the wheelpaths, Figure 6.23(c). Some parts of the higher viscosity mix were also exhibiting these signs, Figure 6.23(d). The surface exhibited a generally black appearance on the softer mix and this may be seen in Figure 6.23(e). Surface deformation and rutting however were not significant, so that the pavement appeared to be satisfactorily stable despite the onset of some flushing.

6.7.2 Performance of Northern Motorway Test Area

Traffic patterns and densities on the Motorway test area had not been established at the time of analysis because the section was a new traffic route. The data for the first 8 months of traffic is presented in Appendix Figure VI.73 but the envelopes of the densification functions are presented in Figure 6.16. Little information can be gained from that per se, except that the usual high initial rate of densification is apparent, there is further evidence suggesting the effect of initial density on trafficked density, and the density is approaching the density given by laboratory-compacted blocks of samples from the plant. There appears to be no significant difference

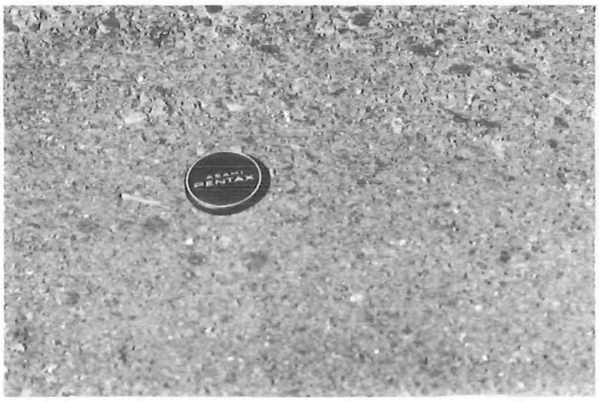


(a) Low Viscosity Mix (8 mths)



(b) High Viscosity Mix (8 mths)

(c) Low Viscosity Mix (21 mths)



(d) High Viscosity Mix (21 mths)

(e) Black Appearance of Low Viscosity Strips (marked)

Figure 6.23 SURFACE TEXTURES OF OVER-COMPACTED MIXES - MAIN NORTH ROAD

6.7.2 (cont.)

in behaviour between the concrete and granular base areas.

6.7.3 Comparison of Testing Track and Main North Road Testing Area

Strip 16 on the Testing Track was a near replica of the low viscosity mix on the Main North Road Test Area and Strips 11 and 13 were near replicas of the high viscosity mix. These replicas in the two different situations permit some comparison of their performance as well as a measure of the acceptability of the Testing Track conditions as representative of normal highway conditions. One difficulty however is that, although they were designed to be replicas, the Testing Track mixes are approximately 1.3 a.v.u. lower than their counterparts in the air voids contents of the compacted samples from the plant. Hence the air voids contents of the Testing Track mixes are expected to be lower than the North Road mixes by that margin.

The low viscosity mix on the track had densified to 1.30% air voids by the end of Stage 2 testing, Figure 6.11(a), and was beginning to show some very slight signs of filaments of asphalt slick on the surface. By the end of Stage 4 when it had densified to 0.87% air voids there was a fair incidence of such asphalt slick and considerable rutting distortion. However, the incidence of slick was not nearly as high as on the North Road, which ostensibly had a higher air voids content, and the filaments were finer. Bad rutting deformation did not occur on the North Road either.

There is a better comparison for the high viscosity mix. On the North Road an area with initial voids of 3.8% densified to 2.8%, Figure 6.15(a), while on the Track, Strip 11 densified from 3.9% to 2.4% air voids by the end of Stage 4, Figure 6.9(a). Another area on the North Road densified from 5.5% to 3.5% air voids, while Strip 13 on the track densified from 7.5% to 3.0% air voids. Once again the Testing Track produced a higher density than normal highway traffic although the difference is not as great as it was for the low viscosity mix. The high viscosity mix on the Track however showed no signs of asphalt slick at any stage.

Data from the Main North Road tend to show slightly more scatter in density and air void content than data from the Testing Track.

6.7.3 (cont.)

There were insufficient observations of stability in the North Road data to form a similar conclusion for stability. However it appears that the greater control of construction and trafficking at the Track compared with the Main North Road Area has resulted in greater uniformity of results. Concern that horizontal stresses on the Track might have been causing abnormal scatter is thus lessened, and any scatter that might be caused in this way is offset by other variations common in the field.

Although the difference in mix properties as produced is probably the major cause of the higher densities on the Testing Track there is still some evidence that the Track is producing slightly higher densities than attained on normal highways. There are three possible reasons:

(i) The sustained period of high temperature on the Track may be a more severe condition than directly applicable to a particular locality. This is unlikely in this case because the 21-month period spanned two summer seasons.

(ii) The slower speed of the Testing Vehicle and consequently longer loading duration probably causes higher densities through increasing creep effects due to viscous flow. The mix behaves more elastically under shorter loading durations.

(iii) The horizontal stresses existing under the Testing Vehicle tyres may cause more squeezing and kneading of the mix thus producing slightly higher densities.

While the effect of each of these causes may be slight for the higher viscosity mix, causes (i) and (ii) will have a more significant effect on the low viscosity mix, as was observed.

The difference in surface textural behaviour was probably due to the number of load repetitions, flexure of the underlying structure and the relative quality control of the mixes.

(i) The greater number of load repetitions on a highway and the higher temperature of the actual pavement surface probably both combine to create a surface crust of high density. In addition to this, engine oil on the pavement will decrease the viscosity of the

6.7.3 (cont.)

binder at the surface. These factors may lead to flushing before the average full core air voids content drops below say 3%.

(ii) The cement-treated base of the North Road pavement was more flexible than the concrete base of the Testing Track. Flexure would produce a compressive strain pulse in the pavement surface that could create the fine longitudinal cracking effect which was observed. Together with a low air voids content near the surface this might produce "free asphalt" which could appear on the surface.

(iii) Quality control of the mix on the North Road was not good. Laboratory blocks moulded from samples taken from trucks on the job showed a variation in air voids of 2.2 to 5.1% for the low viscosity mix. This was probably the result of variations in both binder content and aggregate gradation. Excess binder for the actual gradation would decrease the air voids content and hasten the onset of flushing.

6.7.4 The Influence of Foundation Flexibility

A major difference between the Testing Track and a normal highway is in the flexibility of the foundation. A theoretical analysis showed earlier that vertical stresses and strains differ by only 1-3% between rigid and flexible base conditions cf. 3.3.2. Horizontal stresses and strains however are much greater when the base is flexible.

The results from the Main North Road seem to confirm the theoretical findings. Flexural effects, and their implicit horizontal stresses and strains, had a much more significant effect there than on the Testing Track. The difference in stable state densities between the two situations is probably due to a number of causes more significant than the difference in vertical stresses.

The test area on the Northern Motorway was intended to provide conclusive evidence on this question since it covers both concrete and granular bases. There is nothing in Figure 6.16 to suggest any significant difference in densification properties

6.7.4 (cont.)

between the two situations, and after 12 months there is no visible textural difference.

6.8 THE RELATIONSHIP BETWEEN INTERNAL STRUCTURE AND MIXTURE PROPERTIES

6.8.1 Orientation of Aggregate Particles

It was mooted earlier that the interparticle structure of the aggregate within the mixture is a major factor in determining its strength and deformation under imposed load. It was also mooted that any change in mix properties as a result of work done on the mixture must be accompanied by a change in potential energy of components in the structure. This implied possible changes in the interparticle structure of the aggregate, a characteristic that may be studied by the orientation of particles within the mixture. This pilot study was conducted by the author in order to obtain evidence in support of his thesis on the mechanism of deformation of asphalt concrete. The thesis is postulated in detail in the next section.

Five strips in Test Series 1 were measured for particle orientation characteristics before and after the total trafficking sequence. Any change in characteristics would probably be small but if it were at all significant it ought to be measurable over the full range of testing. A short summary of the characteristics is given in Table 6–II and a discussion on statistical method and a more detailed table are included in Appendix IV. Particles shorter than 2 mm in exposed length have been disregarded in the analysis due to errors of resolution and measurement.

The aggregate particles do exhibit a preferred orientation in the range of 3 to 8 degrees from the horizontal rather than being oriented in a random manner. This is demonstrated by the chi squared (χ^2) test which, being greater than 9.21 in all cases, indicates that there is less than 1% significance of random orientation, i.e. the particles have a preferential orientation. The measure used here of the deviation of individual values is the probability, P, of the orientation of a particle being within 10^o of the mean. For convenience this probability is termed the "probability of occurrence".

TABLE 6-II SUMMARY OF PARTICLE ORIENTATION ANALYSIS

STRIP NO.	TESTING STAGE	MEAN ORIENTATION (degrees)	SAMPLE SIZE	CHI SQUARED	PROBABILITY OF BEING WITHIN 10^{o} OF MEAN
11	0	3.6	920	553	0.326
	4	3.1	786	334	0.298
12	0	6.1	986	594	0.341
	4	4.3	822	420	0.354
13	0	7.3	586	356	0.321
	4	3.1	494	46	0.274
16	0	2.8	1118	432	0.288
	4	5.5	1268	553	0.291
18	0	5.1	1549	744	0.328
	4	5.4	1311	543	0.298

N.B. See Appendix IV for details of analysis. This table summarises analysis for particles with an exposed length $>$ 2 mm.

6.8.1 (cont.)

The normally designed mix with 80/100 pen. asphalt on Strips 11 and 13 was studied to detect any difference in characteristics due to construction density, as well as traffic. Comparing the untrafficked states, Stage 0, of the two mixes shows that the heavily-compacted Strip 11 has a lower mean orientation (3.6°) with respect to the horizontal than the lightly compacted Strip 13 (7.3°), although the probability of occurrence is not significantly different (0.326 vs. 0.321). Higher compactive effort seems to result in flattening the particles down to a more horizontal position. The same trend is noticed from the action of traffic. By comparing Stages 0 and 4 of each Strip the mean orientation is reduced to 3.1° in each case and the probabilities of occurrence near these means are slightly lower. Traffic action thus seems to have a similar compactive effect to construction rolling although the effort has not been strong enough to shift so many particles to the region of the new mean.

The effect of excess binder in the mix may be assessed by comparing the rich mix A on Strip 12 to the normal mix B on Strip 11. The mean orientation is slightly higher for the rich mix (6.1° vs. 3.6°) but the probability of occurrence is slightly higher (0.341 vs. 0.326). Traffic again has the effect of slightly reducing the mean orientation (to 4.3°) and the probability of occurrence is also increased (to (0.354). This illustrates the more fluid nature of the rich mix permitting easier manipulation of individual particles.

The low viscosity binder was used in the normally designed, if slightly rich, mix D but this does exhibit contrary characteristics. The mean orientation increases slightly (2.8° to 5.5°) over trafficking and the probability of occurrence remains moderate (0.29). Since the sample size here was larger than in the previous examples (1100-1200), the conclusion is that variations of this order are to be expected. Hence too much significance should not be attached to the foregoing examples.

The dry mix of the low viscosity binder, on Strip 18, shows an insignificant shift under traffic. The mean orientation remains virtually constant (5.1°, 5.4°) and the probability of occurrence is

6.8.1 (cont.)

similar to foregoing levels although decreasing slightly (0.328, 0.298).

6.8.2 The Importance of the Orientation Findings

The analysis of particle orientation just described is not complete. There are a number of further statistical tests that need to be applied to make the findings fully statistically sound, see Appendix IV for details. Obviously the sample size ought to be increased although there was a large amount of work involved in processing the 13,000 particles of the pilot study. A check on the directional significance of the findings should be made by taking measurements at right angles to the direction of travel.

All this work, however, seems of questionable value since the changes in characteristics are clearly small and difficult to establish. The author believes however that the pilot study has been valuable in establishing that the particles are preferentially oriented and that this structure is dependent on the nature and magnitude of compactive effort. The trends observed in the foregoing subsection are probably correct but the author is reluctant to attach any more significance to them than that.

6.9 CONCLUSIONS AND THESIS

6.9.1 The Trends of Performance Indicators

(i) The air void content of a mix, when related to design properties, is a reasonable though not definitive indicator of performance. It is well related to strength if internal particle structure is kept similar but is often poorly related to deformation without consideration of deformability. The air void content of a pavement is decreased under the action of traffic and it may decrease beyond the predicted value in severe conditions. The inherent scatter in air voids values generally decreases under traffic but some scatter is retained by the effect of initial density and by rutting deformation effects if these are present.

(ii) The measured stability of the core samples bears little relation to the laboratory value, generally being 30-50% less. This is probably due to a difference in internal structure. For the core samples from each mix there is a strong exponential relationship of

6.9.1 (cont.)

both stability and flow with air voids. Thus for a fairly constant internal structure, air voids and stability provide corroborative evidence on compaction. However neither are definitive. Bearing capacity was of no more significant value.

(iii) Evidence was strong that a stable state is achieved in the pavement at lower temperatures. The evidence was less strong, but nevertheless adequate, that a stable state is reached for pavement temperatures above 40°C. The squareness functions used as a test of stable state were unsuccessful both because of their inherent definition and the effect of initial density.

(iv) The measurement of transverse surface profiles, though not highly accurate, provided valuable information on the deformation of the mixtures. Rutting was clearly evident in less stable mixtures. There was a consistent but only fair correlation between the mean decrease in pavement thickness and increase in density.

(v) Surface texture varied considerably and displayed a dependence on binder content. Significant erosion of fines from the surface occurred under traffic. Despite this, skid resistance values remained consistently high for all mixes throughout testing.

6.9.2 <u>The Influence of Loading Conditions</u>

(i) Volume of traffic has a highly significant effect on compaction. A rapid change in indicator occurs during the first 500 - 1000 passes of the Testing Vehicle on the track and during the first 100,000 - 200,000 passes of mixed vehicular traffic on a normal highway. This initial change is often as much as 50% of the total change.

(ii) The effects of load and temperature on a certain mix are interrelated by the viscosity-temperature function. The effects are shown quantitatively in Figure 6.22 for one basic mix. The achieved stable state density increases as viscosity decreases. The increase in stable state density due to an increase of load decreases as viscosity decreases. Thus stable state conditions are more sensitive to viscosity conditions than to load conditions in the common range of pavement temperatures.

6.9.2 (cont.)

(iii) Stable state conditions achieved on the Testing Track under high contact pressure and high temperature conditions were generally close to those predicted by laboratory-compacted samples from the plant. This finding gives strong support to Marshall predictions. It also implies that the Testing Track gives a satisfactory simulation of traffic. Occasionally the stable state conditions on the Track were higher than predicted probably caused by longer loading duration of the slow Testing Vehicle.

6.9.3 Mix Construction and Design

(i) The constructed density of the surface course has a significant effect on the stable state density for most loading conditions. The effect may be of the order of 1 - 3 air void units. Under the combination of heavy load and high temperature however the effect becomes negligible. Low construction densities may thus attain design density under such conditions, but that will probably be accompanied by significant permanent deformation and by the possibility of rutting.

(ii) Surface course thickness has only a slight effect on densification. Thinner pavements tend to densify more slowly and reach a slightly lower stable state than thicker pavements. Thicker pavements however may suffer excessive deformation and distortion.

(iii) Density and stability decrease with depth in the surface course. This is due more to the similar decrease in temperature than to reduction in the vertical stress with depth. Horizontal stresses at the surface may also contribute.

(iv) Differences in design density due to variations in asphalt content are retained in similar magnitude under the action of traffic. Core stabilities however did not retain the differences in design stabilities. A rich mix is susceptible to higher deformations and distortion, and has a much closer surface texture than normal or lean mixes which both behaved satisfactorily.

(v) The effects of asphalt viscosity have been summarised in clause 6.9.2(ii).

(v) The effect of maximum aggregate size is closely related to surface course thickness. Acceptable behaviour was obtained from thicknesses in the range of 2 - 5 times the maximum aggregate size. Thicknesses near the upper limit of this range may deform badly under severe loading conditions and greater thicknesses

6.9.3 (cont.)

generally exhibit large deformations and a tendency to rutting.

(vii) The effect of aggregate shape and gradation was not studied experimentally. They are likely to affect the rate of densification slightly but are expected to have little other effect per se.

6.9.4 Correlation Between Track and Highway

(1) The loading conditions of the Testing Track tend to produce a slightly higher stable state density than is obtained on a normally trafficked highway, although this depends largely on the specific temperature and traffic characteristics of the locality.

(ii) The longer loading duration and slower speed of the Testing Vehicle greatly increases the rate of densification and very slightly increases the stable state density.

(iii) Poor transverse distribution of trafficking by the Testing Vehicle in the preliminary and first series of tests caused abnormal rutting deformation. When the distribution had been corrected, deformations were as uniform as on a normal highway.

(iv) Horizontal stresses under the tyres of the sharply-turning Testing Vehicle seemed to cause some horizontal kneading and movement when mix viscosities are low. The stresses would also increase any erosion that might occur.

(v) Flexure of the underlying pavement structure causes horizontal surface stresses in a highway pavement which may cause fine longitudinal cracking and slight flushing as occurred on the Main North Road test area. Such effects were hardly noticeable on the corresponding Testing Track strip. Flexure does not appear to affect the rate or the magnitude of densification, although evidence on this is limited.

6.9.5 A Postulated Mechanism of Traffic Compaction

On the basis of the knowledge from the earlier chapters of this thesis and the findings from this chapter, it is possible to postulate a mechanism by which traffic compaction occurs. At this stage, the postulate is based on observations of properties which are of a permanent or semi-permanent nature, as discussed in this chapter. Consideration of transient response is made in the next chapter.

When an asphalt concrete mixture is laid by a paving machine it essentially consists of a number of irregularly-shaped solid particles randomly distributed and oriented within a viscous fluid matrix. Construction compaction forces these aggregate particles into a more closely-packed structure. The highly regular nature of the repeated compactive forces together with the constriction of the upper and lower layer boundaries cause these particles to be preferentially oriented in a near horizontal position. (The upper layer boundary, i.e. the pavement surface, is constricted in the locality under consideration by the stresses existing over the tyre contact area, cf. 3.2.3, 2.2.3.). The fluidity of the matrix at low viscosity permits this much movement. At this stage, the mix has developed a structure strong enough to carry the compactive load with only little deformation but the low binder viscosity makes it still susceptible to distortion by a greater loading intensity.

At normal pavement temperatures, under the action of normal vehicular traffic, the internal mix structure will adjust to resist the imposed stresses if it is not already sufficiently stable. If construction compaction has been light, the internal structure will not be highly developed and traffic action will have a similar effect to construction compaction in moving the particles closer together and possibly reorienting some of them. At high binder viscosities, the stresses due to vehicular traffic may be sufficient only to move the particles closer together without any reorientation. In order to translate and rotate a particle the imposed stress must be sufficient to overcome the viscous resistance of the binder film. This resistance may be high because of high binder viscosity or because of a thin film thickness as in a lean mix. Hence any particular particle may be considered to possess an energy limit that must be exceeded before it can be moved. The energy limit may be exceeded by increasing the load or the limit itself may be reduced by increasing the temperature. A stable state is deemed to exist for conditions that do not exceed the minimum energy limit in the mixture.

6.9.5 (cont.)

Hence, the reason why a mix may have the same effective stability at different densities lies in the structure of the mix. A rich mix having a thicker binder film around the particles will provide less viscous resistance to compacting stresses, and thus densifies more rapidly. A mix will tend to deform considerably and distort under uneven loading if the viscous resistance is too low to maintain the interparticle structure. This may happen when the air void content is low if there is excess binder because the binder film will be thick and viscous resistance consequently lowered. It is apparent therefore that the film thickness and viscosity of the binder are the primary factors in compaction of the mix by traffic. Since the Marshall "flow" value is sensitive to these two factors it may be regarded as a valuable indicator to the extent of compaction by traffic.

The study of particle orientation provided some confirmation of this thesis although the changes measured were very small and not always significant. The author considers his thesis sound despite such low significance because the orientation analysis was subject to a number of statistical and experimental errors.

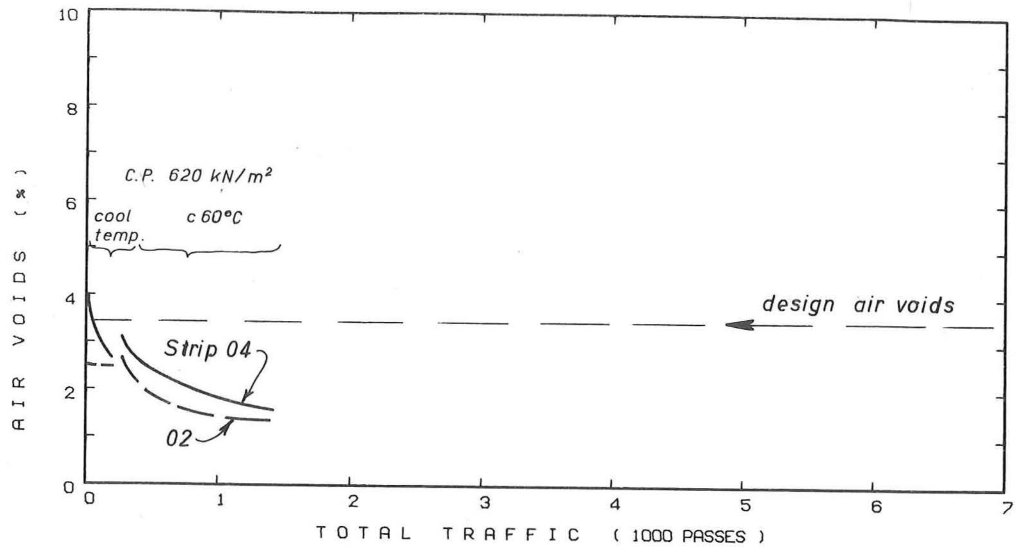


Figure 6.7 RESULTS FROM FIRST PRELIMINARY TESTS

All Strips Mix D
(Low viscosity , optm)
Thickness 50 mm

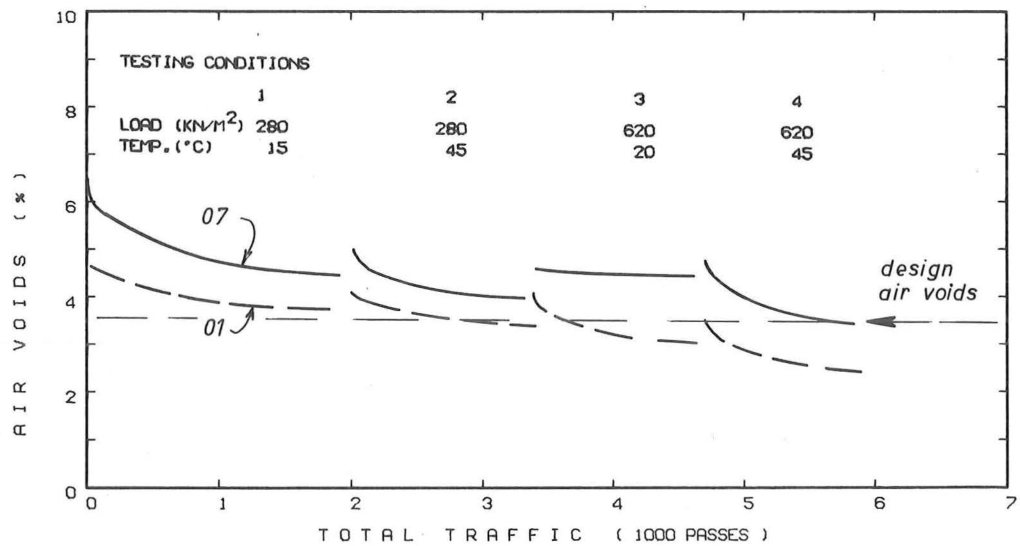


Figure 6.8 RESULTS FROM SECOND PRELIMINARY TESTS

*(Please fold out)

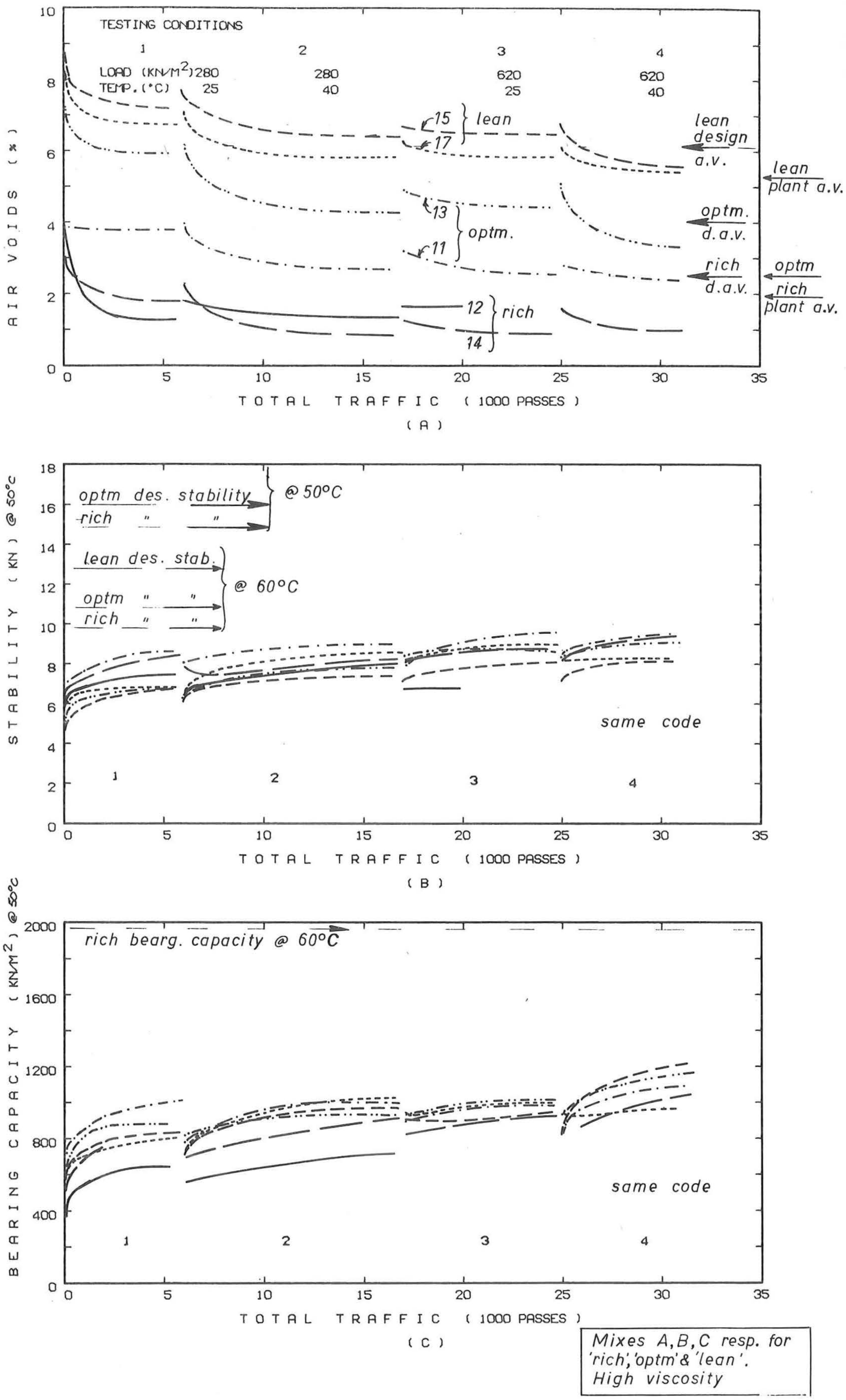
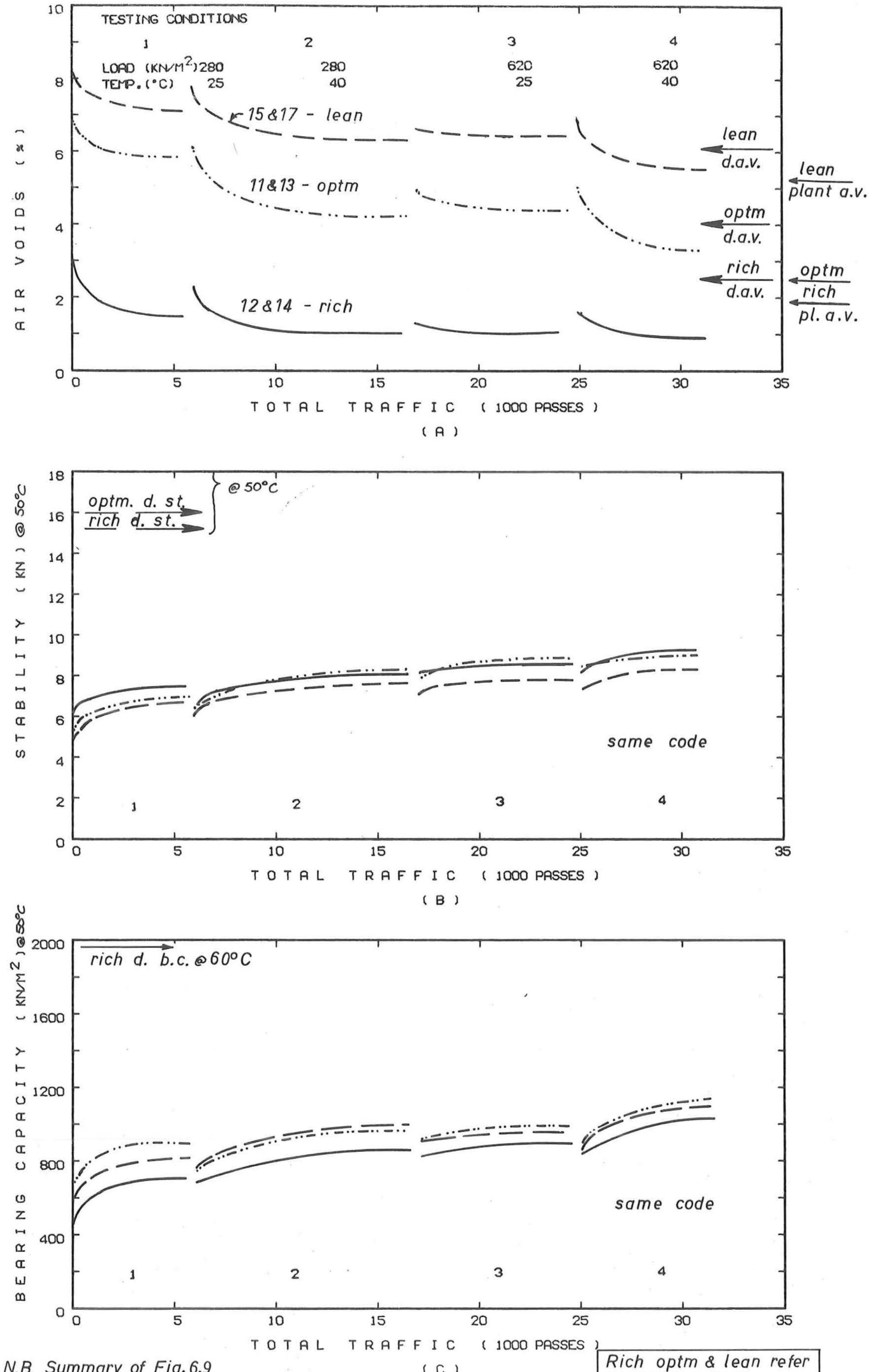


Figure 6.9 *EFFECT OF BINDER CONTENT AND INITIAL DENSITY — SERIES 1*

(Please fold out)



N.B. Summary of Fig. 6.9

Rich optm & lean refer to Mixes A, B, C resp.
High viscosity; 16mm agg

Figure 6.10 EFFECT OF BINDER CONTENT AND DESIGN AIR VOIDS — SERIES 1 (Please fold out)

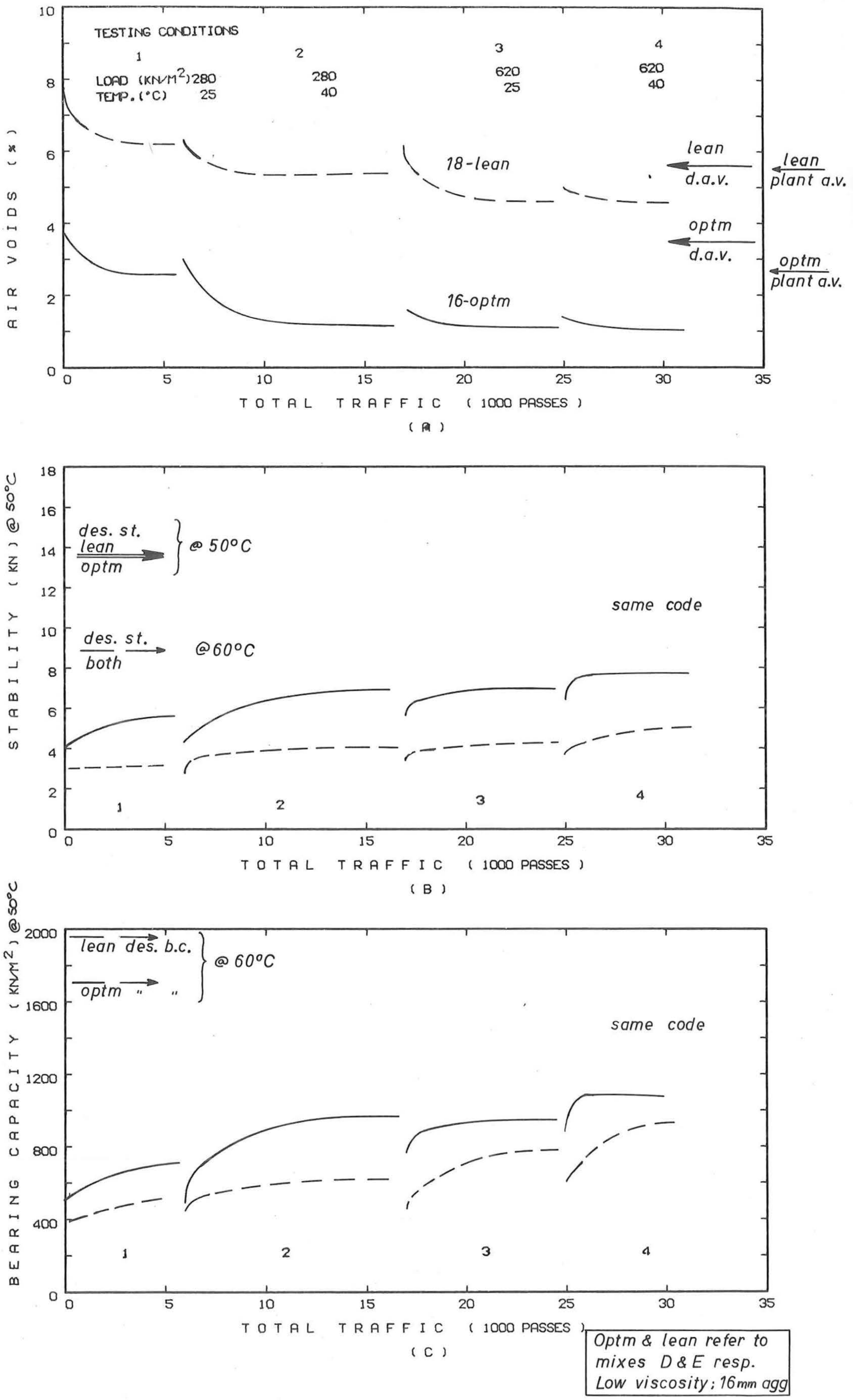


Figure 6.11 EFFECT OF BINDER CONTENT FOR LOW VISCOSITY ASPHALT

(Please fold out)

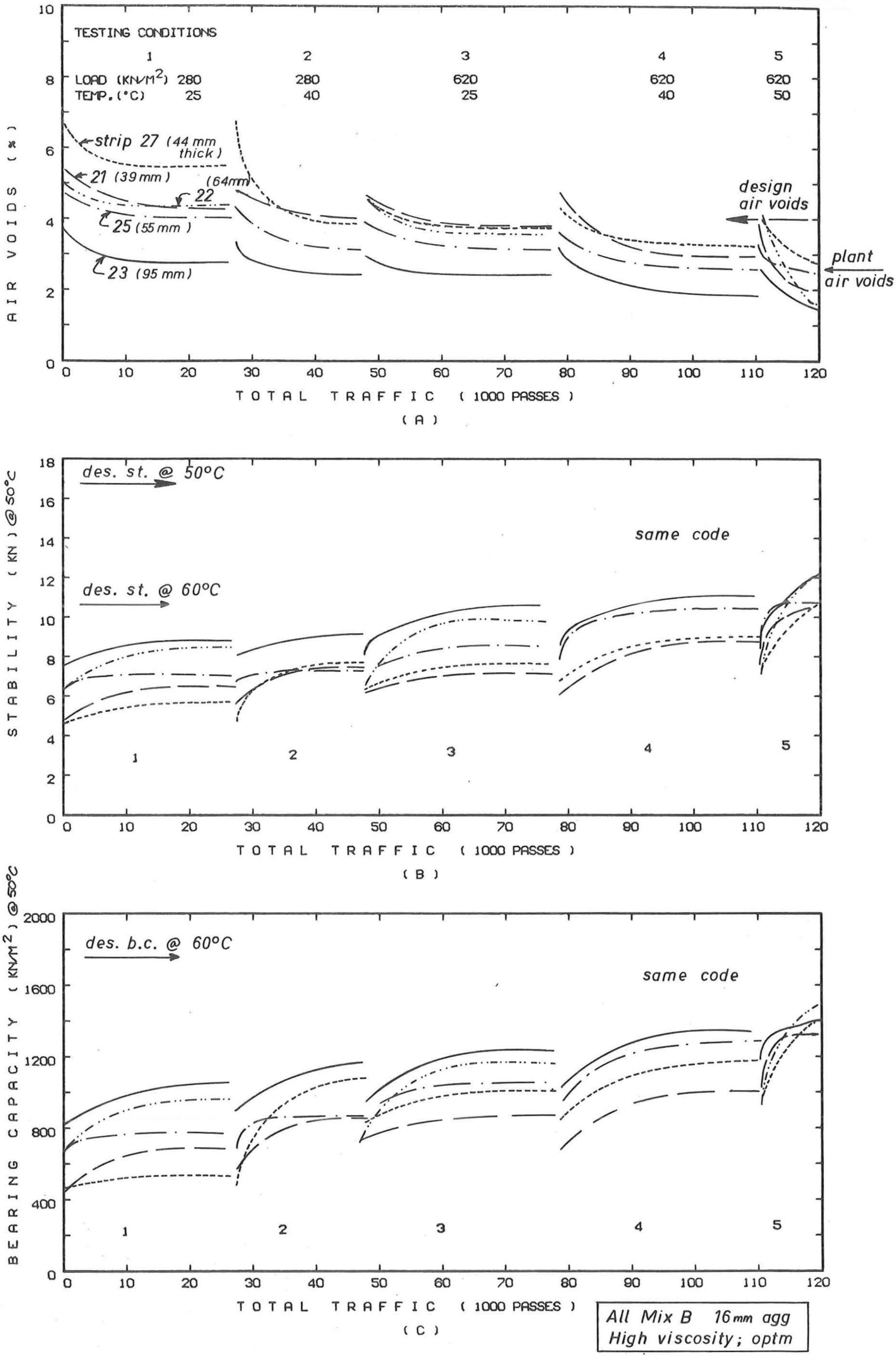


Figure 6.12 EFFECTS OF LAYER THICKNESS AND INITIAL DENSITY – 16 mm AGGREGATE

(Please fold out)

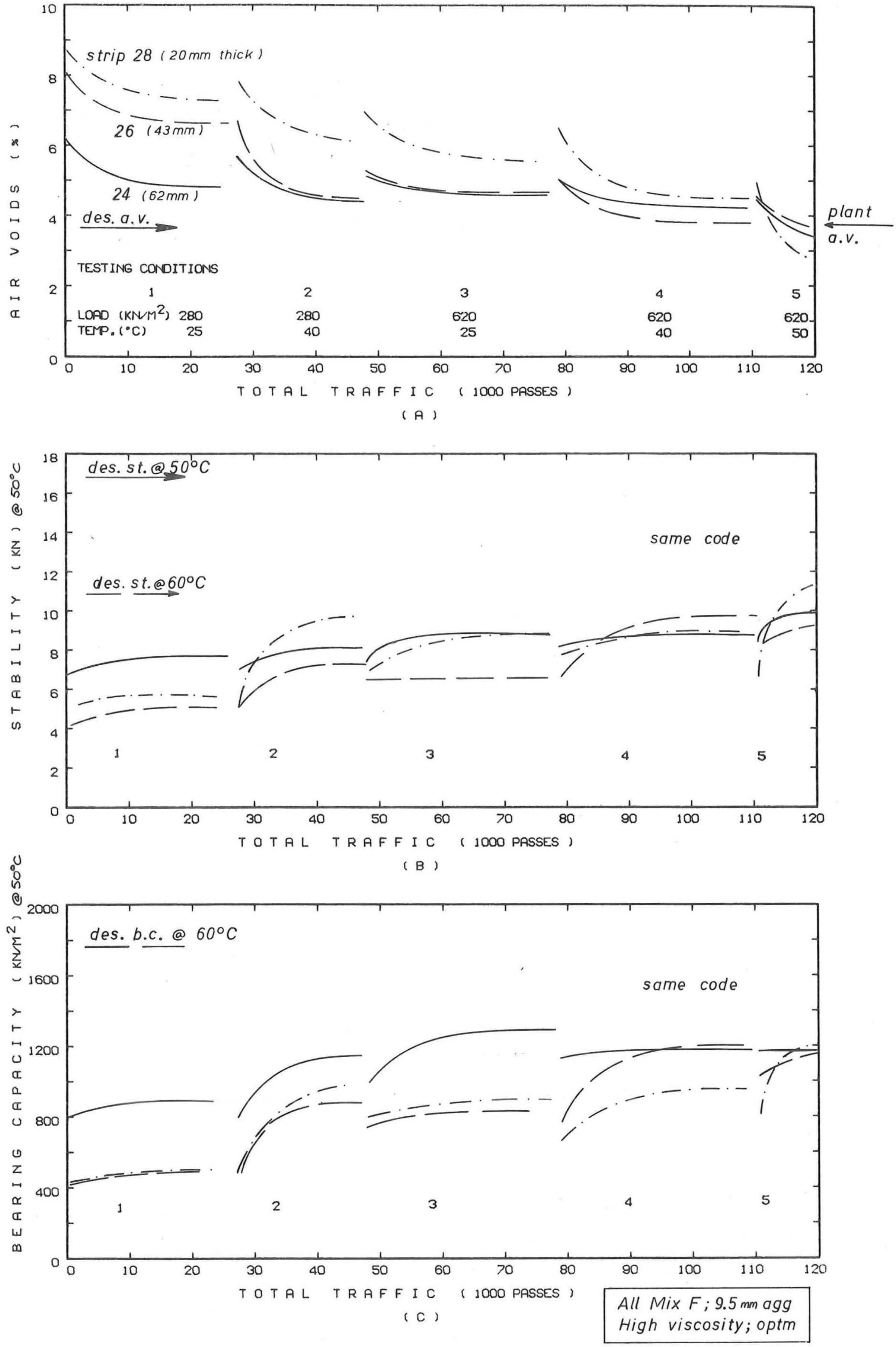
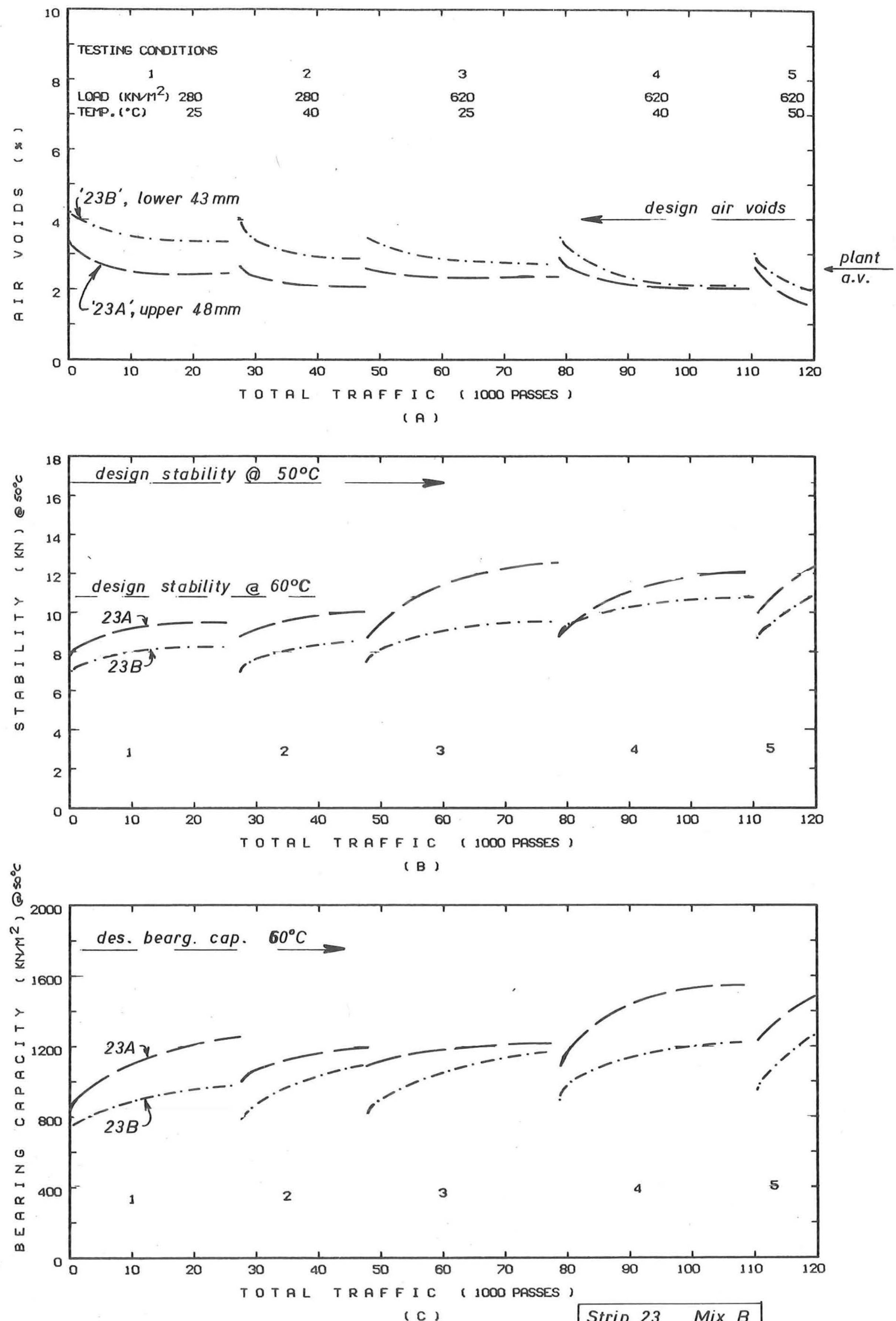


Figure 6.13 EFFECTS OF LAYER THICKNESS FOR 9.5mm AGGREGATE

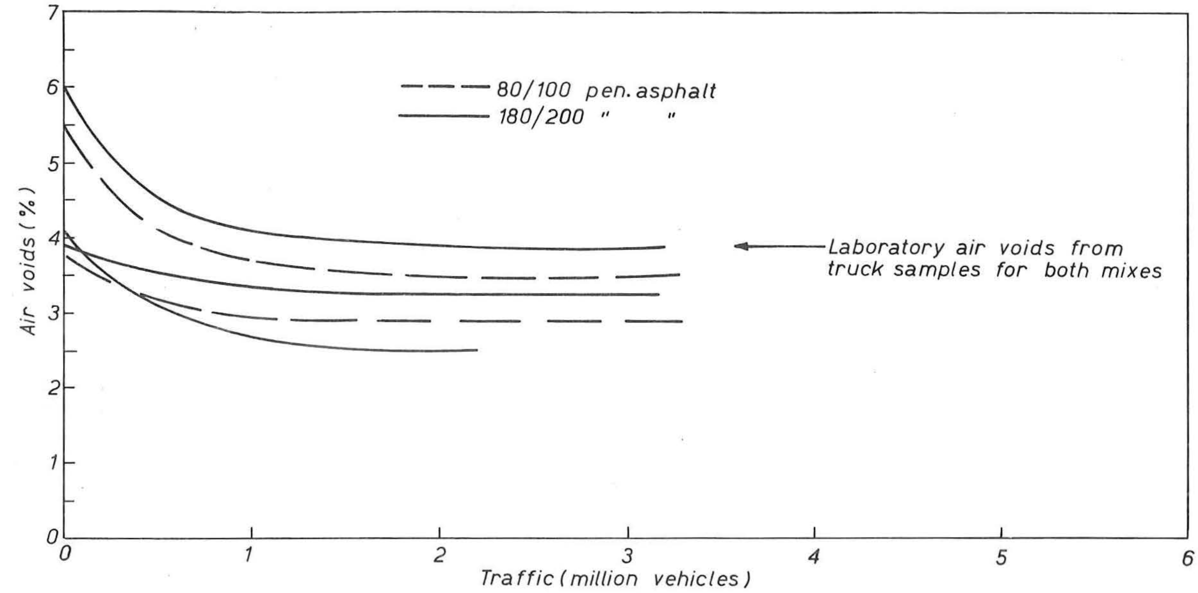
(Please fold out)



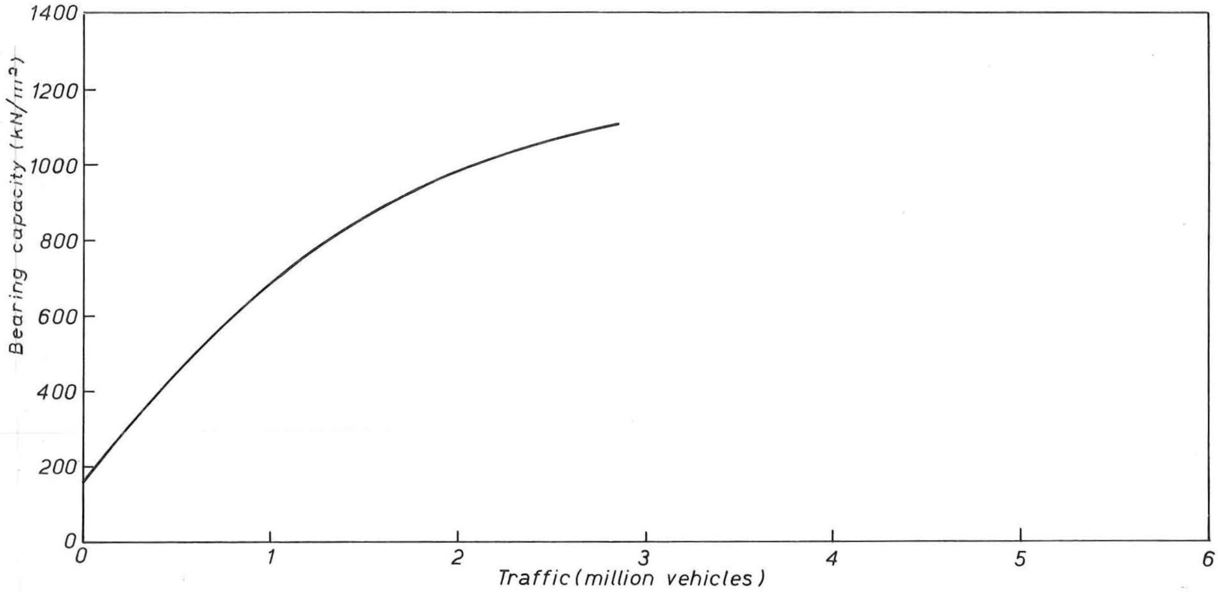
Strip 23 Mix B
Thickness 95mm

Figure 6.14 <u>*VARIATION OF INDICATORS WITH DEPTH*</u>

(Please fold out)



(a) Representative densification functions including different initial conditions



(b) Representative bearing capacity function (N.B. Lane traffic volumes have been equalised in these figures)

Fig. 6.15 DENSIFICATION FUNCTIONS FROM MAIN NORTH ROAD TEST AREA

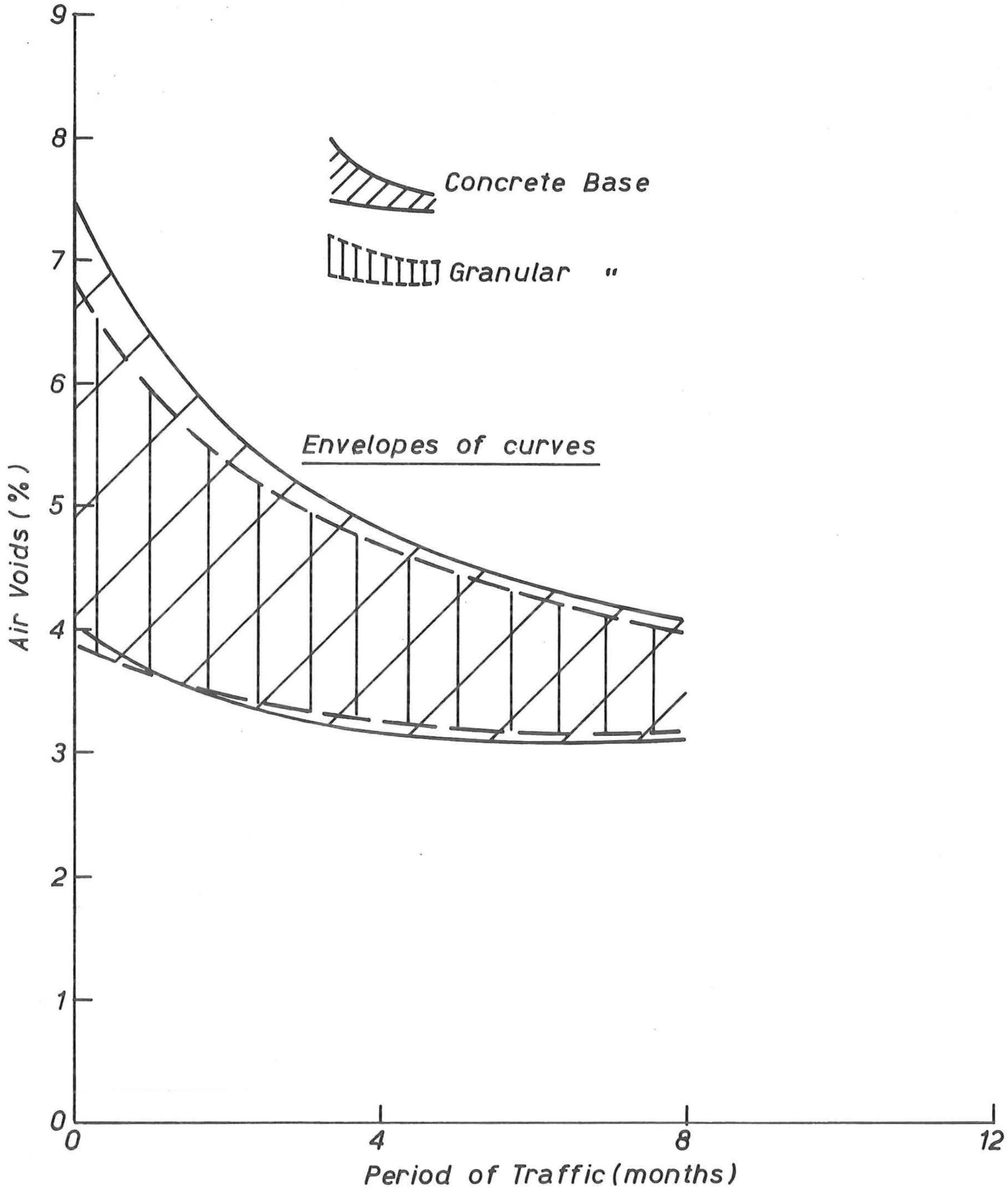


Fig. 6.16 DENSIFICATION FUNCTIONS FROM NORTHERN MOTORWAY TEST AREA

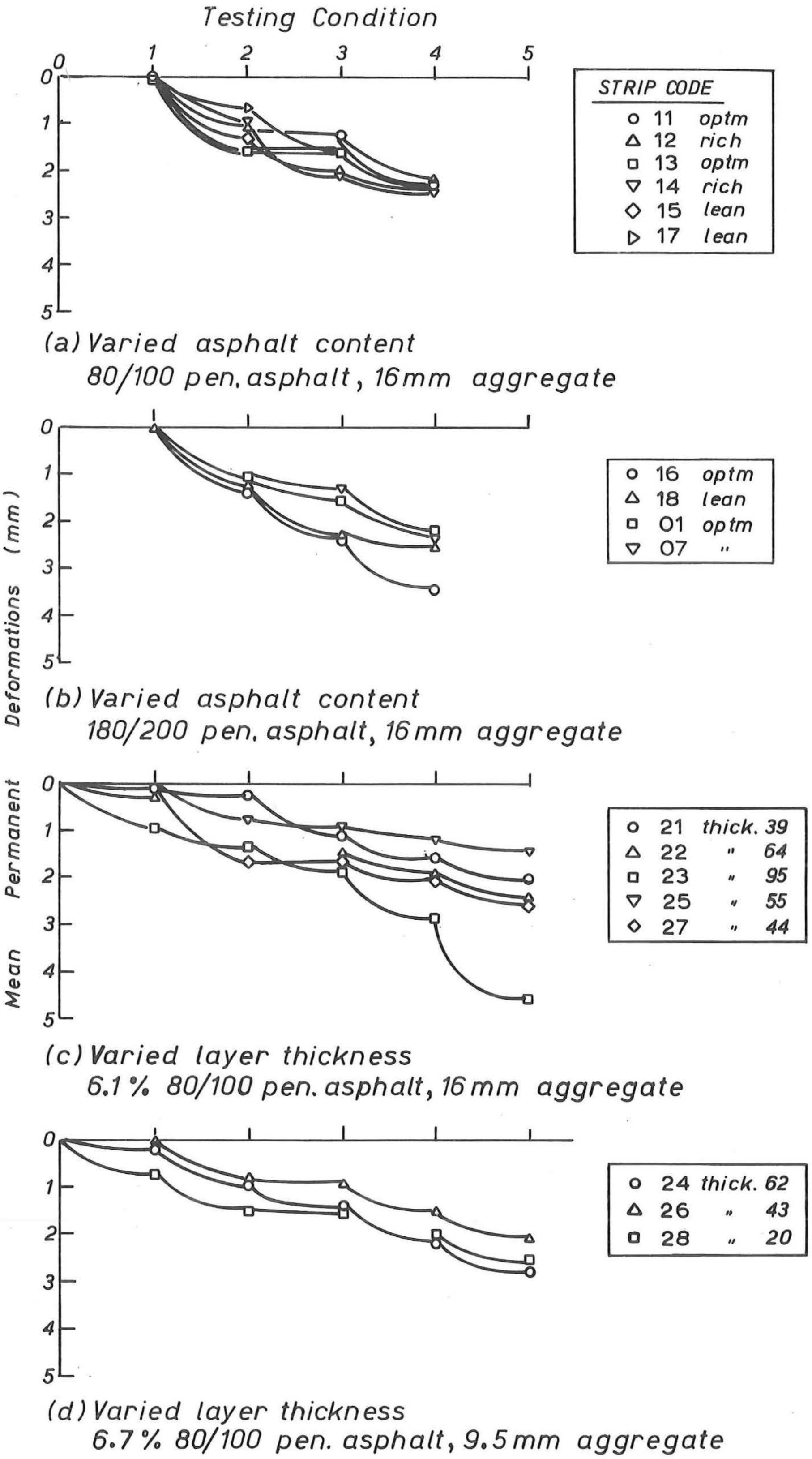


Fig. 6.17 COMPARISON OF PERMANENT DEFORMATIONS FROM TESTING TRACK

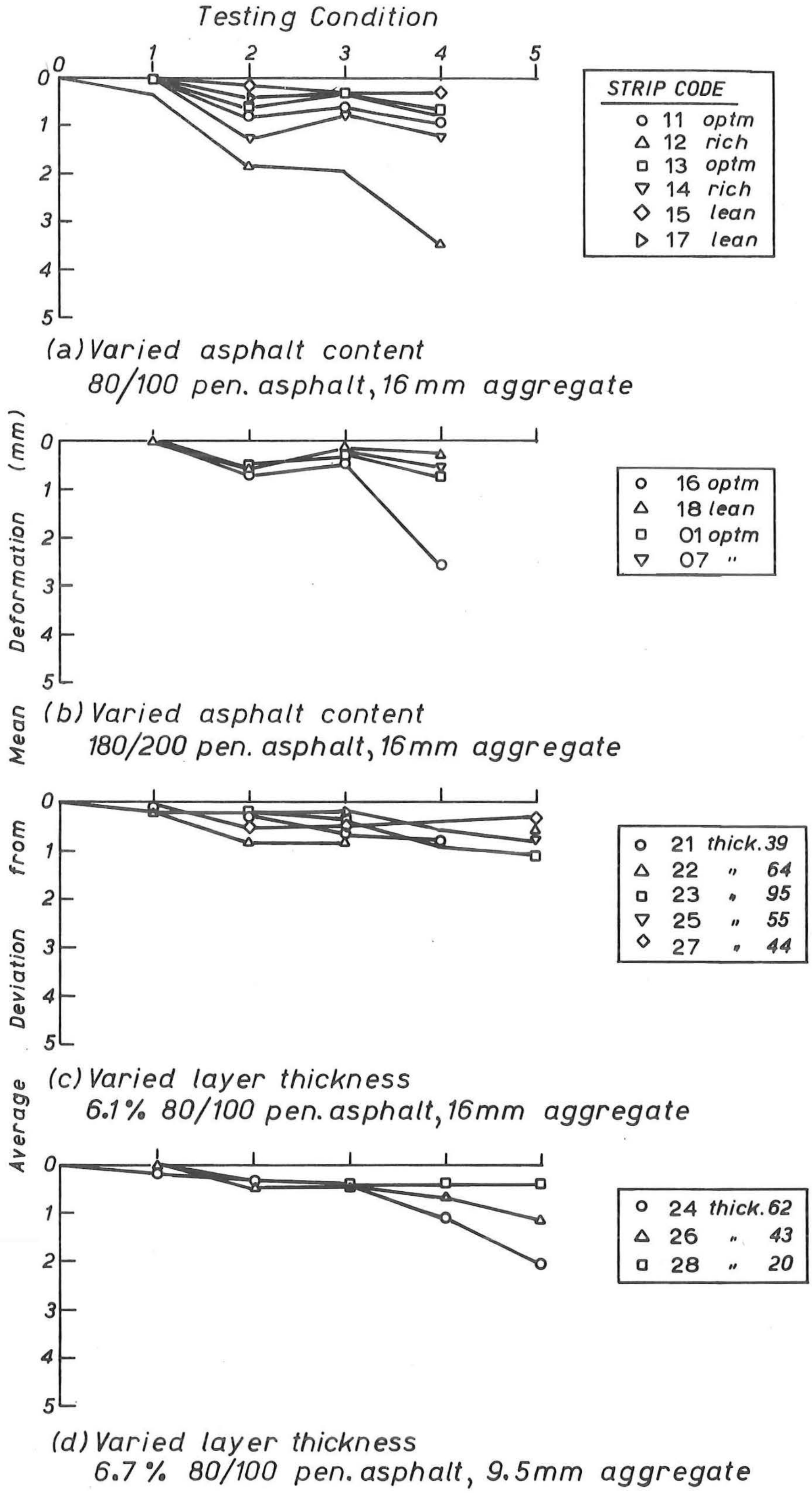
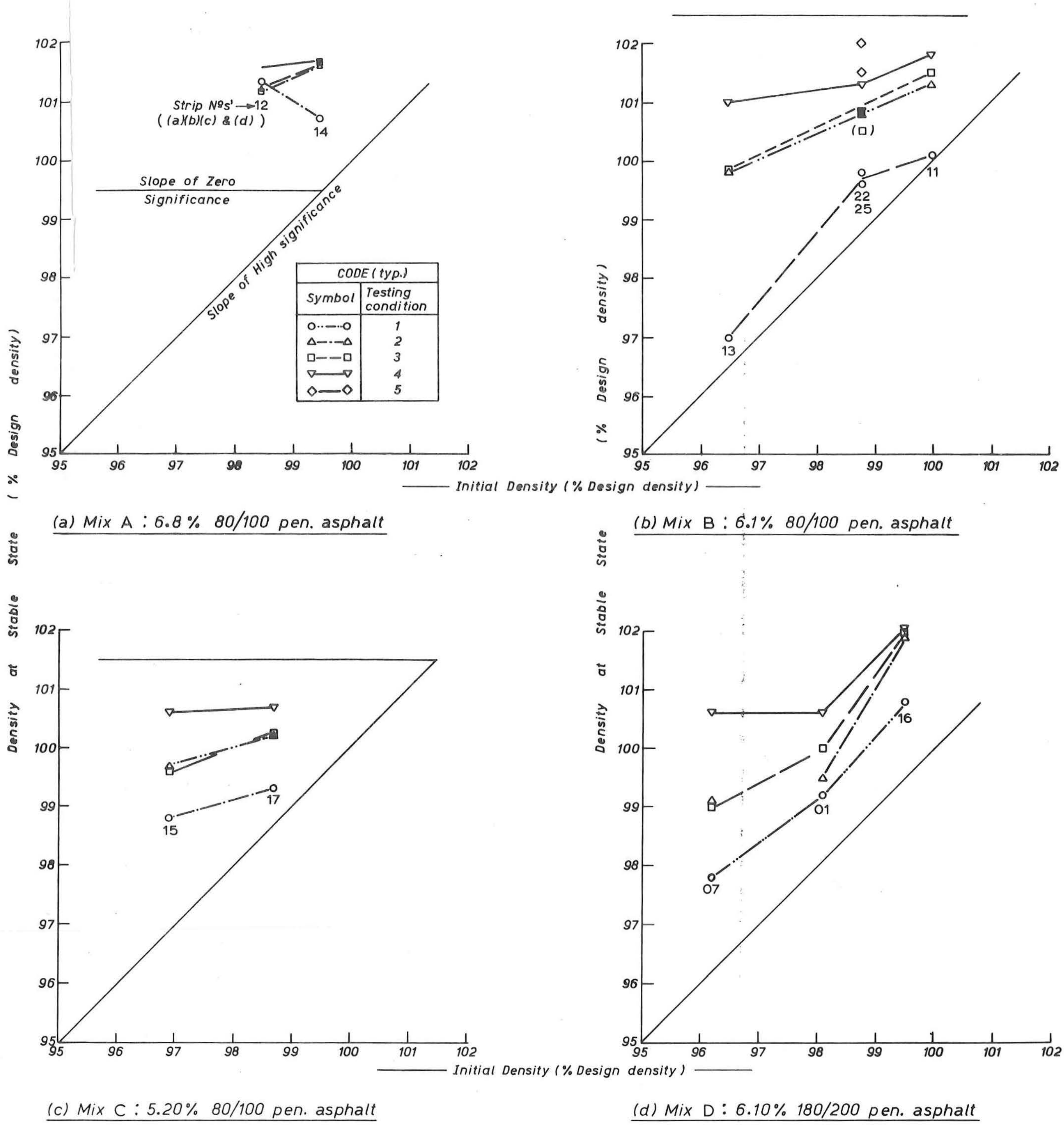


Fig. 6.18 COMPARISON OF DEVIATION FROM MEAN PERMANENT DEFORMATIONS FROM TESTING TRACK.



(a) Mix A : 6.8 % 80/100 pen. asphalt

(b) Mix B : 6.1% 80/100 pen. asphalt

(c) Mix C : 5.20% 80/100 pen. asphalt

(d) Mix D : 6.10% 180/200 pen. asphalt

Fig. 6.19 EFFECT OF INITIAL DENSITY ON STABLE STATE DENSITY

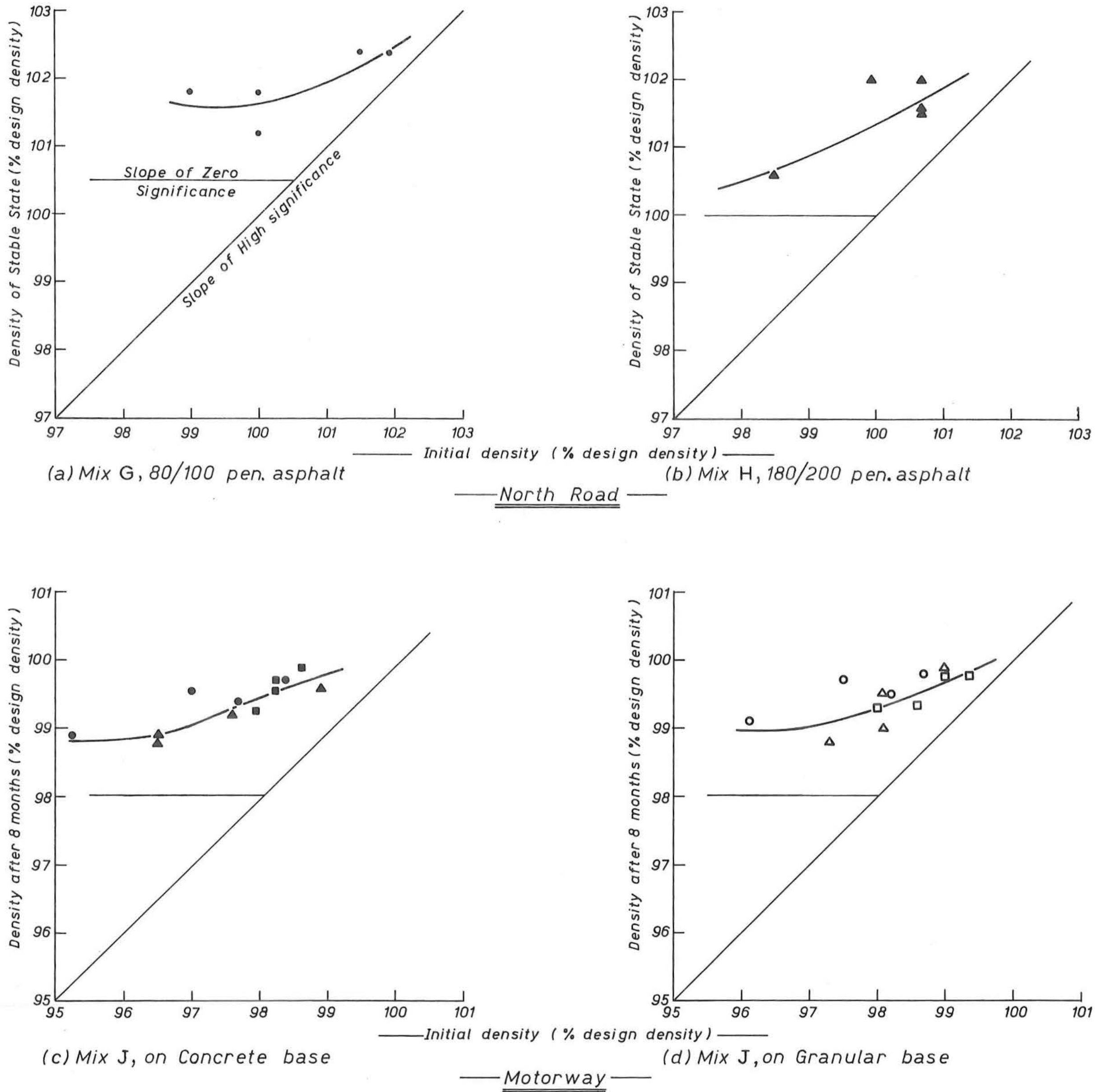


Fig. 6.20 EFFECT OF INITIAL DENSITY ON STABLE STATE DENSITY —— HIGHWAY DATA

CHAPTER SEVEN

THE TRANSIENT RESPONSE OF THE SURFACE COURSE TO TRAFFIC

Measurements of transient deformations caused by a moving wheel load were made in the vertical, longitudinal and transverse directions. The characteristics of the time profile of these deformations and their dependence on load, pavement properties and temperature are discussed. This leads to the observation of a significant dependence of dynamic stiffness on temperature and pavement layer thickness.

7.1 SCOPE OF OBSERVATIONS

7.1.1 Measurement Technique

Many techniques have been attempted for the measurement of stress and strain in soil materials[113]. Strain measurement generally has less inherent problems than stress measurement and is of greater interest and value for bituminous materials at this stage. Strain techniques so far employed on bituminous materials have included bonded resistance gauges fixed to a horizontal interface[81] and leaf spring gauges[79]. However only the latter technique has been able to measure vertical strains and even then indirectly. A more promising and direct technique involves the use of a pair of small disc-shape coils arranged to give a signal in proportion to the change in spacing of the coils. The instrument was developed in Illinois by Truesdale et al. between 1962 and 1965 and has recently been marketed commercially. The technique was adopted for this dynamic study and the author has described at length the principle, calibration and his use of this technique, including the method of placement, in another publication[114].

7.1.2 Location of Gauges for Track Tests

Transient strains were measured using 25 mm diameter coil strain gauges. At least three gauges were placed in each strip of the Testing Track for measurements in the vertical, longitudinal and transverse directions. An exception to this was Strip 28 which was too thin to accommodate horizontal gauges. The gauges were placed near the centre of each strip and on the centreline of the traffic path so that they received maximum trafficking, see Figure 7.1(a). The vertical, longitudinal and transverse gauges were set 150 mm apart to prevent interference, ib (b), and a thermocouple was placed at middepth alongside each gauge. Two of the thicker strips, 23 and 24, had intermediate coils placed in the vertical gauge to enable observation of the depth profile of deformation, ib (c), (d). There were two non-standard vertical gauges: on Strip 21 where the base coil failed during paving, ib (e), and Strip 28 which was too thin for an axially aligned gauge, ib (f).

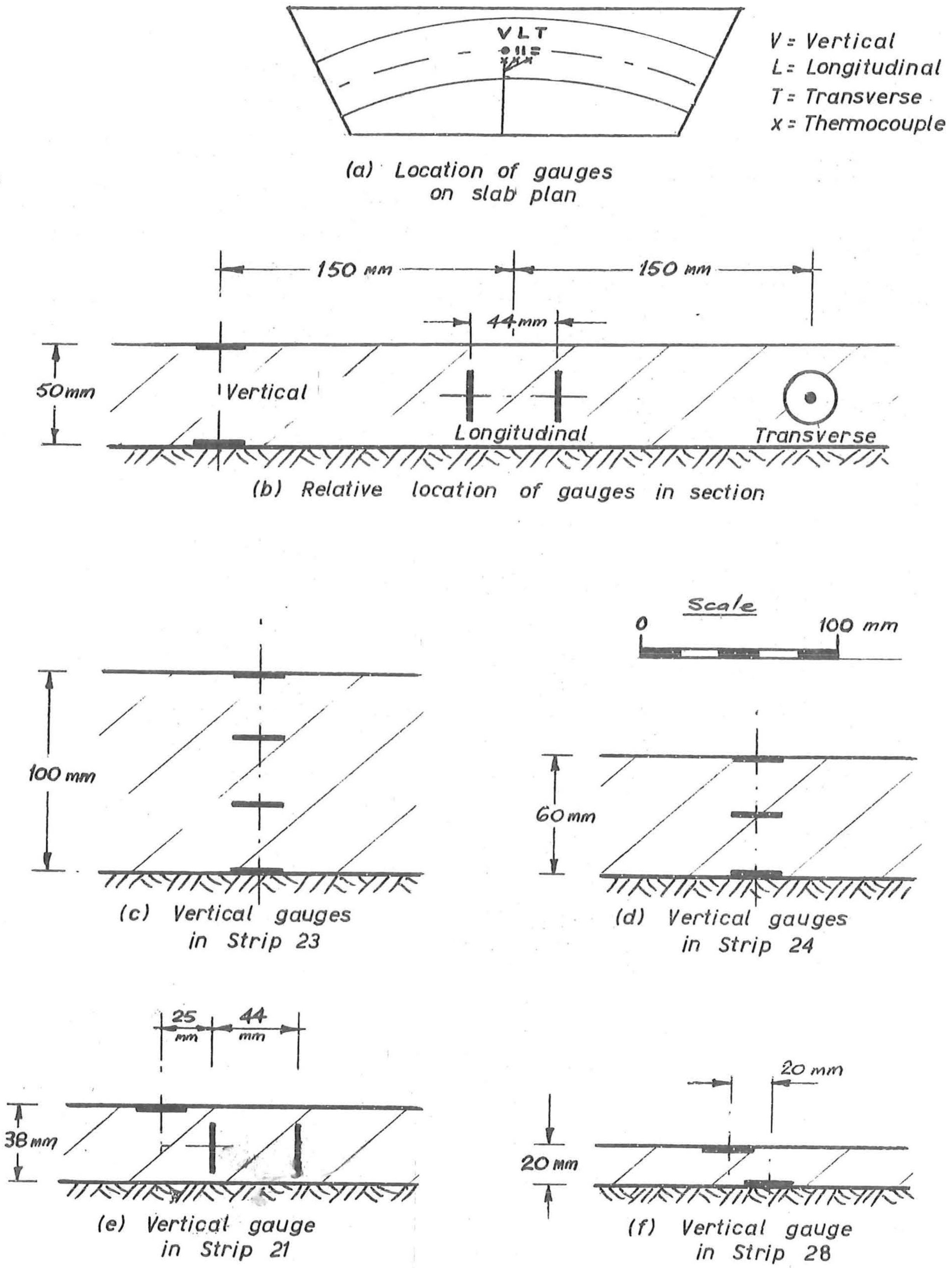


Figure 7.1 LOCATION OF STRAIN GAUGES IN TRACK TEST SERIES 2

7.1.3 Observational Procedure for Track Tests

Transient observations were taken from each gauge at the beginning and end of each testing stage, and permanent strain measurements were made from the amplitude setting on the instrument at null balance. Transient readings were made on a portable high speed ultra-violet-light chart recorder, shown shrouded by black cloth in Figure 7.2. The strain bridge was used at its highest sensitivity for all readings apart from a number of readings at high temperatures.

The continually changing motion of the testing vehicle made recording of tyre placement over a gauge difficult. After some experience of the pattern of vehicle movement this could be done with a fair degree of accuracy but the exact location could not be determined so a minimum of four or five readings was selected. The facility for fixing the centre of motion of the vehicle was used in the last stage of the test since it was found that this caused the gauges on four alternate strips to be trafficked nearly centrally by one or other of the tyres. This facility was not used for any of the earlier measurements because of the drastic rutting influence of each wheel following a fixed path.

As placed, most of the gauges performed well although several of them ceased functioning during the high temperature testing of Stage 5. The most likely cause for a gauge becoming inoperative is failure of the joint of the lead wires to the coil. Excessive movement of the matrix material in the vicinity of the coil may cause such a failure during service[115]. The author had thought this was unlikely for asphalt concrete but clearly it can happen at high temperatures or in soft mixes. Apart from the failure during paving of the base coil on Strip 21, which has already been mentioned, the only other coil to be unusable from the time of placement was the lower intermediate coil on Strip 23 and this was due to misalignment.

7.1.4 Use of Gauges in Highway Tests

At the same time as the placement of gauges on the track for Test Series 2, gauges were also placed on the Northern Motorway Test Area during the surface course construction phase.

Nineteen gauges were embedded in the strips as indicated on the plan shown earlier in Figure 5.11. They were located in the

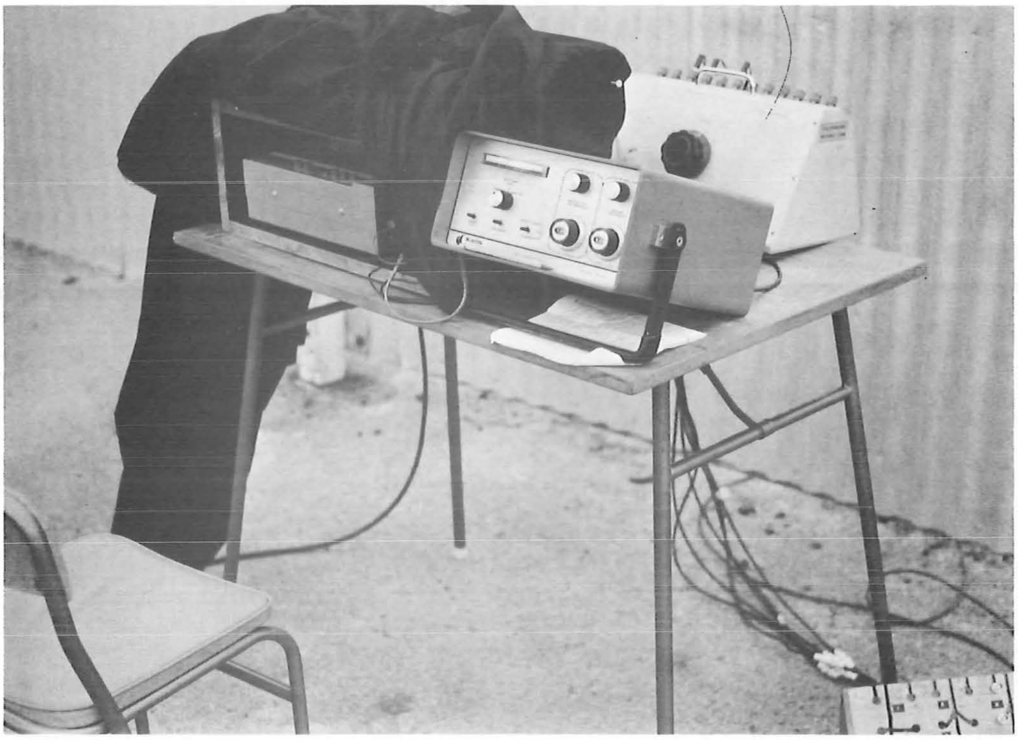


Figure 7.2 <u>*STRAIN MEASURING APPARATUS AT THE TESTING TRACK*</u>

Figure 7.3 <u>*STRAIN MEASUREMENT ON NORTHERN MOTORWAY*</u>

7.1.4 (cont.)

wheel-paths nearest the edge of the pavement to facilitate a safe connection to recording instruments without excessive cable lengths. At least one vertical gauge was placed in each lane of each strip with extra ones and two horizontal gauges, transverse and longitudinal, in the two medium-compacted strips. Thermocouples were placed alongside each vertical gauge. Four vertical gauges were found to be inoperative after paving.

Strain profiles from the gauges were measured on the same equipment as used at the test track, the recorder being housed in a blacked-out car. A loaded truck with an 80 kN single rear axle was employed to provide the standard wheel load. The truck was driven at about 20 km/h over the gauge position which was heavily chalked: the exact transverse location of the truck tyre over the gauge was recorded photographically, Figure 7.3.

Strain measurements were taken at 0 and 8 months. Suitable weather was needed to obtain some approximation to the preferred standard of 25^{o}C but the choice of times was severely limited by the need to pre-arrange vehicles, etc. Hence the temperatures at the time of measurement generally departed from the standard.

7.1.5 General Considerations

Interpretation of the observations will depend on the interaction of the gauge coils and the asphalt concrete matrix, and on cognisance of what deformations the gauges are actually measuring.

The vertical gauges measured the total vertical deformation over the pavement depth. Expressed on a unit length basis this could be termed the "average vertical strain over the depth of the surface course." The use of the term "strain" may be misleading in its pure sense when applied to a non-homogeneous mixture of solid particles in a viscous fluid. Hence the term will be used throughout this chapter to denote "average strain" (a.s.) unless specifically indicated otherwise.

Interaction of the rigid coil discs with the asphalt concrete matrix is expected to be satisfactory at low temperatures when the matrix is very stiff. At high temperatures or in a soft matrix the discs will probably act as large flaky particles in the mixture.

7.1.5 (cont.)

Because the discs are at least 50% greater in diameter than the maximum stone size, they tend to have an averaging effect on local deformations and so measure the average strain.

Disturbance of the matrix in the vicinity of the gauge was minimised in the placement technique. For the vertical gauges it was confined to less than 3% of the gauge length, and for horizontal gauges it was usually of the order of 15% of the gauge length. Although the latter figure sounds high, the use of a stiff asphalt mastic (40/50 pen., 3 mm maximum aggregate size) ensured a realistic distribution of strain to the coil discs.

Any alteration in the seating of the top coil of a vertical gauge was achieved during the first few vehicle contacts and so, with the very low degree of disturbance, the accuracy of vertical measurements should be high. Interference of the vehicle type with the surface coil should be negligible since the gauge is rather insensitive to any horizontal or rotational displacements that the tyre may cause. The sensitivity of the signal to lateral displacements is less than 2% of the sensitivity to axial displacements for displacements up to 1% of the gauge length, and a one degree rotation of one coil gives one hundredth of the signal of a 1% axial displacement[114]. For the same reasons, the horizontal measurements will be insignificantly affected either by the flexural curvature of the whole pavement or by any abnormal vertical, i.e. lateral, displacement that might be caused by the vehicle tyre traversing a filled slot.

Each gauge was calibrated individually before use, op. cit, and hence estimation of average strain should be accurate to the degree of resolution. With the instrument at maximum sensitivity the noise amplitude was equivalent to approximately 30×10^{-6} a.s. with a maximum level of 70×10^{-6} a.s. and so the degree of resolution depended on the magnitude of the signal. For measurements of the order of 100×10^{-6} a.s. resolution was better than $\pm$ 10%, although for smaller measurements resolution might be as poor as $\pm$ 20%.

7.2 TREATMENT OF DATA

7.2.1 Abstraction of Dynamic Data from Chart Images

A curve representing the significant signal was drawn on the trace by visual estimation in order to separate the signal from the noise, see Figure 7.4. This could be done within the degree of resolution mentioned in the previous section. Measurements of the signal were taken at the points indicated to within 0.2 or 0.5 mm depending on the noise and signal amplitudes. The signal amplitude on the chart traces ranged from 1 to 60 mm. The time intervals were measured to approximately 2 milliseconds.

The average measurements from each group of four or five traces were then multiplied by calibration factors to obtain the average strain.

Corrections for the location of the tyre over the gauge were made linearly on the basis of the contact pressure profile for strain characteristics and on the basis of length of tyre contact area for time characteristics. These corrections were only applied to the Motorway data and they were based on the photographic record of wheel location, see Figure 7.3.

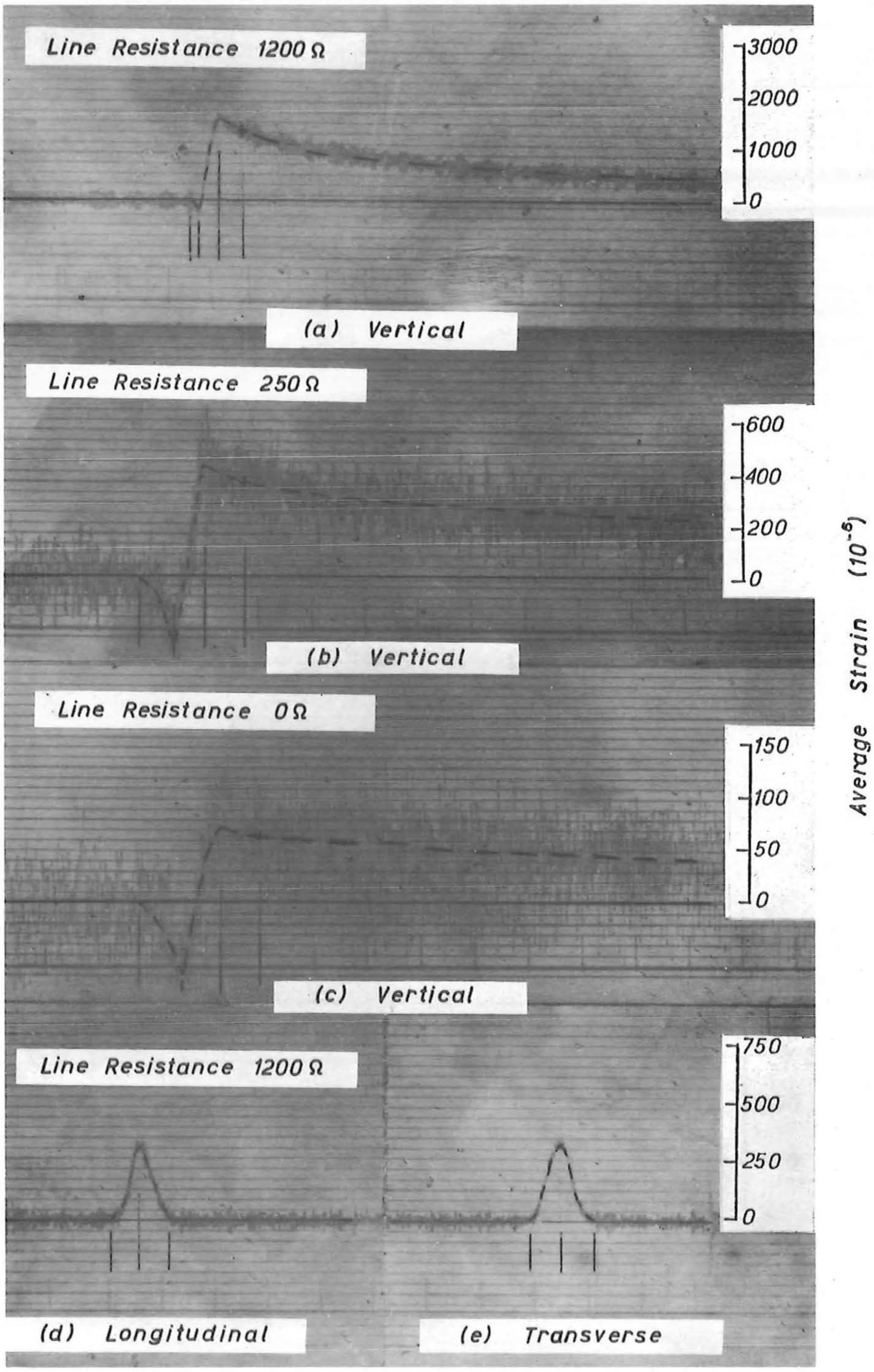
The data are summarised in Appendix VI, Tables VI-III to V.

7.2.2 Reduction to Terms Independent of Load

In order to compare the dynamic characteristics under each loading condition, it was desirable to reduce each characteristic to a term independent of load.

Gusfeldt and Dempwolff[81] established a linear relationship between horizontal strain and tyre contact pressure, and Brown and Pell[83] reduced horizontal strains to a "normalised" form by division by the tyre contact pressure. Hence it seems reasonable to "normalise" horizontal strains in the same way as the latter authors. This was done for all horizontal measurements and is shown in the Tables as "Normalised Average Strain" (N.A.S.) in units of 10^{-6} per kN/m^2, i.e. 10^{-9} m^2/N.

In a similar way all the peak vertical strains have been reduced to a ratio with contact pressure. There is apparently no reported experimental evidence of vertical strains being proportional to imposed stress but it seems a reasonable assumption with supporting evidence of the kind found by Gusfeldt and Dempwolff, op. cit.



N.B. Ticks on horizontal axes represent time intervals of 100 ms

Gauge coil diameter 25 mm
Maximum Sensitivity

N.B. The records are not interrelated.

Figure 7.4 TYPICAL RECORDS OF TRANSIENT DEFORMATIONS

7.2.2 (cont.)

Andersson[79] reported a less than one to one ratio of peak compressive strain to load but that was with constant inflation pressure so the central contact pressure would have not increased proportionally with load. The assumption is realistic in terms of elastic behaviour but it is not so realistic in viscous behaviour where the load-deformation characteristics are non-linear. The initial rate of compression leading to the peak compressive strain is very high[79], however, and for that brief period behaviour will be predominantly elastic. Brown and Pell, op. cit., reduced vertical strains to a normalised form, presumably on this basis. The author, however, desiring to find a single term indicative of resistance to compaction, introduces a term which is the inverse of normalised vertical average strain, i.e. the ratio of tyre contact pressure to peak vertical average strain. Because of the similarity of such a ratio to a modulus of deformation, the author termed the ratio the "Apparent Dynamic Modulus" (A.D.M.), viz:

$$\text{Apparent Dynamic Modulus} = \frac{\text{Tyre Contact Pressure}}{\text{Peak Vertical Average Strain}} .$$

The word "apparent" is included in the term for three reasons:

(i) The modulus is a unidirectional value not directly related to the three-dimensional stress and strain conditions.

(ii) The tyre contact pressure at the surface is used in lieu of the average vertical stress over the gauge length. Elastic theory, however, indicates that the average vertical stress is within 5% of the contact pressure.

(iii) The modulus is not a characteristic solely of the material but is also influenced by other pavement properties such as layer thickness.

The values of apparent dynamic modulus are included in the aforementioned tables. However, the calculation of the value is not always direct for reasons mentioned in 7.3.1.

7.3 CHARACTERISTIC PROFILES OF TRANSIENT STRAIN

7.3.1 Transient Vertical Strain

The profile of transient vertical average strain has a

7.3.1 (cont.)

characteristic shape apparent in nearly all the traces illustrated in Tables VI-IV and V and this shape is reproduced in Figure 7.5(a).

A slight extension occurs immediately before the tyre contacts the measuring location and reaches its peak in approximately 40 to 50 ms. It is probably a poisson ratio effect. The peak is clearly defined and was adopted as the time origin because the beginning of the extension is frequently ill-defined.

The extension is followed by a period of rapid compression lasting 40 to 45 ms for the light wheel load and 50 to 60 ms for the heavy wheel load and culminating in a usually well-defined peak. The period corresponds to the passage of the tyre contact area over the gauge which, by calculation from the vehicle speed of 18.7 km/h and contact area lengths of 216 and 273 mm respectively, is 42 ms for the light load and 53 ms for the heavy load. The rapid increase in compression throughout the contact period implies (i) that the strain profile lags the surface stress profile significantly, and (ii) that the deformation is not wholly elastic because no plateau is reached equivalent to that in the stress profile. A number of tests at slower speeds and low temperature showed a significant decrease in the rate of compression and only a slight flattening near the peak.

There was no discernible consistent effect of temperature on the shape or the period of this portion of the curve. Unfortunately a direct measure of the effect of load at constant temperature was missed during the interim between testing Stages 2 and 3 at the Track.

The maximum compression peak is followed by a short period of decompression at a rate approximately equivalent to the earlier compression rate. This appears to be a phase of "immediate" elastic recovery and is deemed to end at the point of maximum curvature where the decompression rate drops to a lower value. In some traces this phase is not apparent and this usually seems to occur for tests at the low temperature on strips which have reached a stable state, e.g. Strip 22, Stages 12, 30, 32; Strip 24, Stages 12, 30, 43, 51A, 51B, etc. The exception does not always occur for those conditions

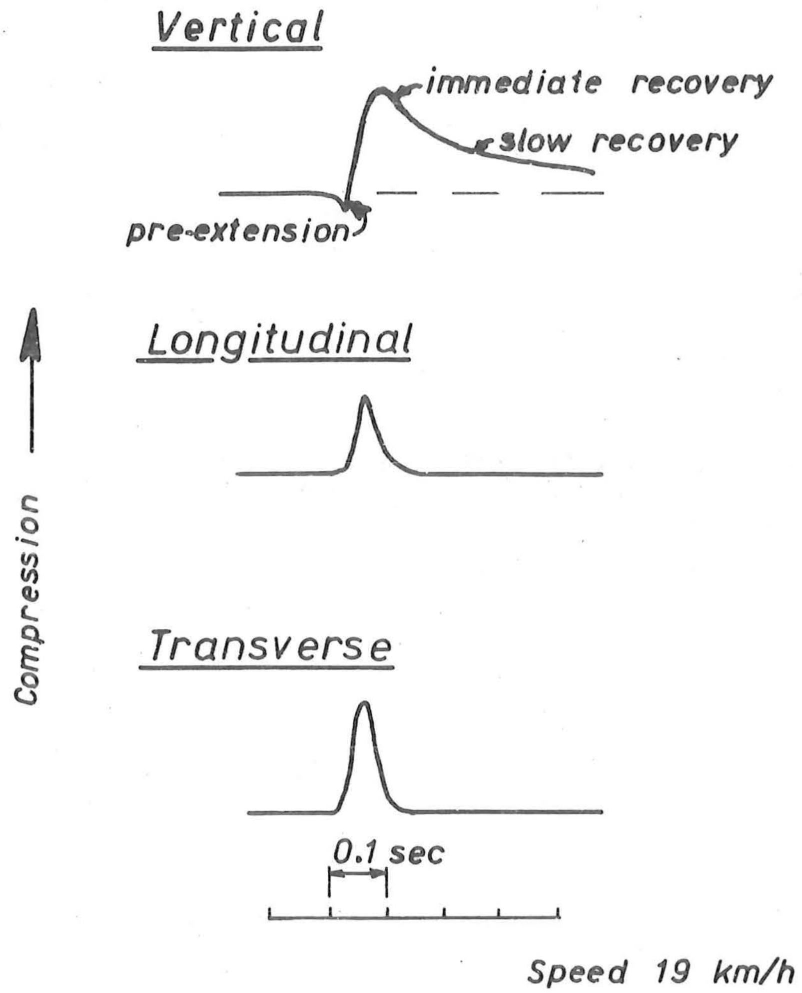


Figure 7.5 CHARACTERISTICS OF STRAIN PROFILES

7.3.1 (cont.)

however, e.g. Strip 26, Stages 12, 30, 32, and the phase is always present under the stable state conditions at the high temperature. This may imply that the conditions at the high temperature are not a really stable state, as suggested in Chapter Six, and it may imply that this phase is dependent on the effective viscosity of the binder film. The latter implication is preferred to the former. The magnitude of the decompression, if present, does generally decrease under the action of traffic so that the phase is probably not elastic or otherwise it would be preserved or even increased under traffic action. A more promising explanation of this is that it is the poisson ratio effect at the trailing end of the tyre contact area superimposing an "extension" on the compression.

A phase of decompression at a much slower, and sometimes nearly constant, rate follows. This phase frequently lasted several seconds and it was only occasionally recorded because of the expense of the light-sensitive chart paper. Usually the signal had returned to zero within 5 seconds before the passage of the next wheel load. However at the high temperature this period was sometimes longer than 10 seconds so that the pavement might receive another loading before complete relaxation depending on the tracking of the Testing Vehicle. When this happened, the peak compressive strain became progressively higher in a compounding fashion under subsequent loadings. This would result in very high shear strains in the binder film probably causing movement of aggregate particles and, in severe circumstances, causing general pavement distortion such as rutting. This indicates that at high temperatures on a highway, a rapid succession of loading, such as by tandem axles, would have a strong compactive influence and could contribute to rutting if the material was unstable under these conditions.

No direct measure of the residual permanent strain due to the passage of a wheel load was obtained due to the duration of the last decompression phase. An indirect measure is gained later from consideration of the accumulated permanent strain at the end of each stage.

7.3.1 (cont.)

The profiles obtained from the Motorway Test Area showed similar characteristics. The time characteristics are different for the initial tests however because the speed of the testing vehicle was too slow. From the profiles, the speed appeared to be approximately 9 km/h, i.e. half the speed of the Testing Track Vehicle. This was rectified in the 8-month observations.

Comparison of the measured profiles of vertical average strain with the predictions of elastic analysis illustrates the disparity due to the dynamic and viscoelastic characteristics of the real material. Figure 7.6(a), (b) shows the symmetrical compressive pulse predicted by the elastic analysis. It is preceded and succeeded by a small extension arising from poisson ratio effects. While the preceding extension appears to be similar to the measured profile, the compression rate is nearly vertical and the profile is nearly square in distinct contrast to the characteristics of the measured profile. The poisson ratio extension at the trailing end of the contact area in this elastic analysis corresponds to the reduction in compression in the measured profile.

One important characteristic of the measured vertical profiles remains to be discussed. The initial extension prior to contact of the gauge by the tyre is often of comparable, or greater, amplitude to the succeeding compression, e.g. on the Testing Track, Strip 21, Stages 12, 22; Strip 22, Stages 12, 40, 42, 43, 51B, etc., and on the Motorway, all vertical gauges at 8 months. This seems to occur for most observations of small deformations in the region of 100×10^{-6} a.s. This is contrary to expectations and elastic theory predicts an extension of only 1 to 3% of the peak compression. Are these large extensions actually occurring or are they the result of a complication in the mensuration technique? A number of possible causes were investigated.

(i) The Poisson ratio effect is very small at low temperatures when the mix is very stiff and is always significantly less than the effect of direct compressive load. Hence this does not account for such relatively large extensions.

(ii) Rotational or lateral displacement of the surface coil

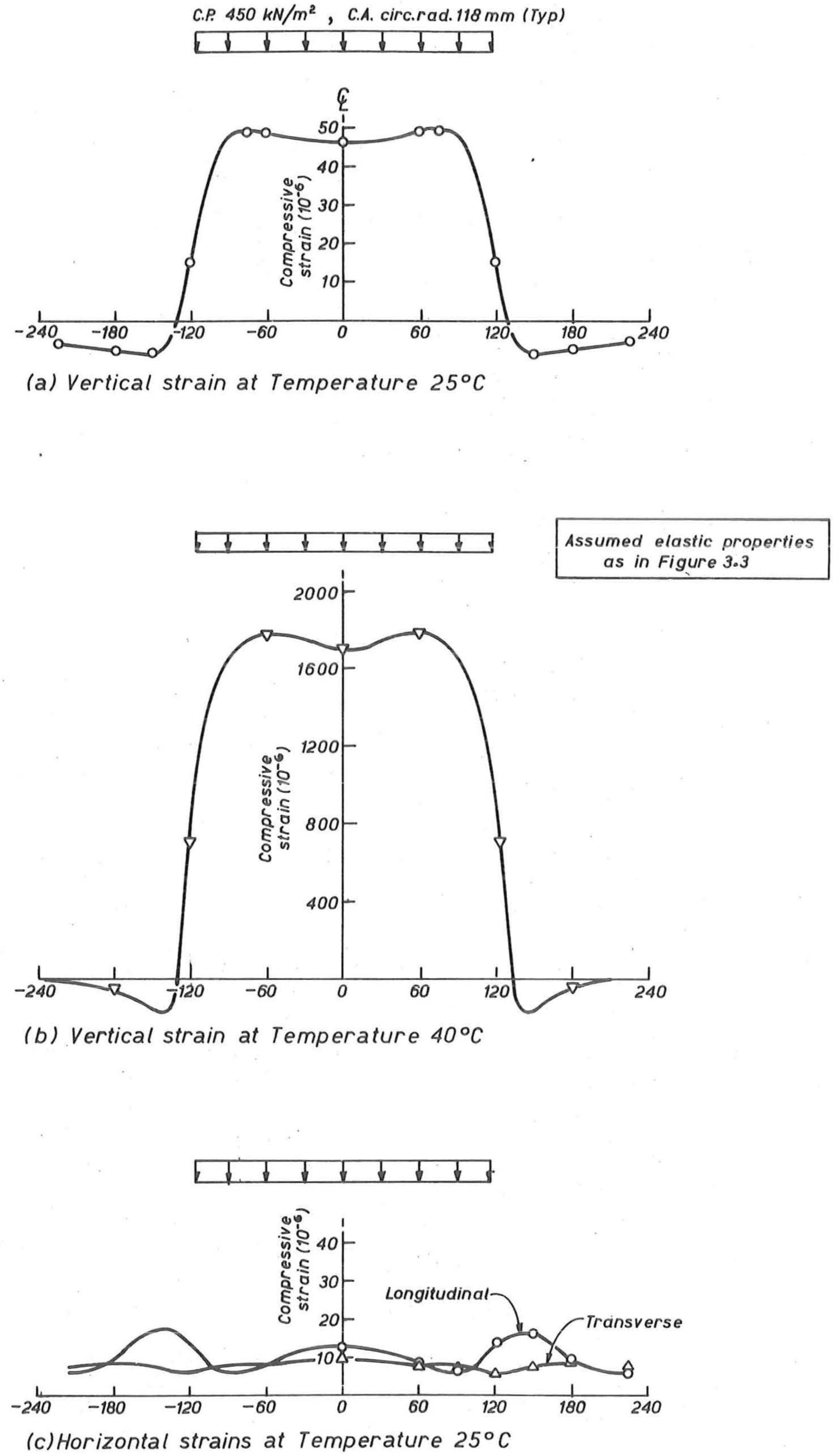


Fig. 7.6 ELASTIC ANALYSIS OF LONGITUDINAL PROFILE OF SURFACE COURSE STRAINS

7.3.1 (cont.)

such as would occur under the action of horizontal surface stresses, is an unlikely cause since these displacements would be small in comparison to vertical displacement and also because they produce less than one twentieth the signal of the equivalent vertical displacement. Furthermore, the lower vertical gauge of Strip 24 which has no surface coil shows some significant extensions.

(iii) The vicinity of the large metallic mass of the testing vehicles was thought to be an unlikely cause because it was considered to be at a safe distance from the gauge, 180-200 mm i.e. more than two to four times the gauge length[114]. However a check was made to determine the precise influence of metal because of the high instrument sensitivity being used. At a distance of four times the gauge length from the centre of the gauge the metal caused a signal deviation equivalent to 0.2 meter divisions, i.e. approximately equivalent to $10-20 \times 10^{-6}$ a.s. for most of the gauges being used. The observed function is shown in Appendix Figure VI.74. This deviation was negative for the vertical gauges indicating an apparent extension, and positive for the horizontal gauges indicating an apparent compression.

The latter explanation thus appears to be the cause of the deviate results. It imposes significant limitations on the application of the technique to surface courses because the magnitude of the effect varies with gauge length and pavement thickness. The effect will be negligible for a maximum gauge length equivalent to approximately 1.8 times the coil diameter or rather less if small vehicles with small wheels are involved in the observations. This effect has been taken into account before calculation of the peak characteristics in the Tables. For vertical strains allowance was made by taking 90% of the range between peak extension and peak compression as the modified peak compression. This is reasonable because the deviation is a constant signal throughout the pulse.

7.3.2 Transient Longitudinal Strain

The characteristic profile of transient longitudinal average strain is shown in Figure 7.5(b). It is usually a single peaked

7.3.2 (cont.)

compression of simple and near symmetrical shape. The leading face is generally slightly steeper than the trailing face with the peak occurring after approximately 45 ms in a pulse of 100 ms duration. The duration is approximately double the tyre contact duration.

The magnitude of the compression peak is frequently as high or higher than the associated vertical compressive strain. At low temperatures it ranges from 100×10^{-6} a.s. in the 38 mm thick strips to 700×10^{-6} a.s. in the 95 mm thick strip, and higher at high temperatures. A small part of this, up to approximately 20×10^{-6} a.s., will be caused by the proximity of the metal of the testing vehicle but it will be less in the thicker than in the thinner pavements. Comparison with the predictions of elastic theory in Figure 7.6 shows that these measured peak strains are approximately ten times greater than predicted elastic strains. The only promising explanation for this is the influence of the high horizontal stresses which exist over the interface of the tyre contact area, section 3.2.3. It does seem unlikely however that this would account for such a big increase especially when its influence ought to diminish with depth and so some doubt remains on this question.

No evidence was observed of a slight extension before and after the pulse as reported by Gusfeldt and Dempwolff[81]. A reversal of strain was noted however during passage of the driving wheel, viz Strip 23, Stage 40 and Strip 27, Stage 22, 51B. There was one exception to these two basic forms and that was a profile with two compressive peaks for Strip 26 Stage 51C.

7.3.3 Transient Transverse Strain

The characteristic shape of the transverse average strain profile was similar to that of the longitudinal profile, a simple single-peaked compression of similar magnitude, Figure 7.5(c). The peak transverse compression was generally slightly higher than the peak longitudinal compression, but the time characteristics were generally the same. This latter characteristic is in contradiction to the report by Gusfeldt and Dempwolff of a pulse duration three times as long as the longitudinal pulse duration.

7.3.3 (cont.)

The profiles from Strip 26, Stage 5 are abnormal and some difference dependent on whether the wheel is driving or free wheeling is noted.

7.4 CHANGES CAUSED BY TRAFFIC ACTION

7.4.1 Apparent Dynamic Modulus

The apparent dynamic modulus increases under the action of traffic and increasing load intensity, Figure 7.7(a), (b). This illustrates the extent to which traffic compaction increases the stiffness of the mix and hence its resistance to deformation. If that figure is compared with Figure 7.8(a), (b) of the corresponding Marshall core stability, it can be noted that the A.D.M. increases much more rapidly than core stability, N.B. the A.D.M. is plotted on a logarithmic scale. This demonstrates the highly significant directional influence of traffic compaction because the mix is developing resistance primarily in the vertical direction and only secondarily in the horizontal direction.

The A.D.M. however is not a direct measure of resistance to traffic action because there is no characteristic value applying to all strips for the stable state conditions of any testing stage. The A.D.M. is also dependent on other properties. Figure 7.9(a), (b) shows how it varies with the air voids content on each strip. Once again the A.D.M. is shown not to be a characteristic solely of the mixture since there is no single characteristic curve for each mix design. Inter-particle structure may be having some influence on it and so may layer thickness which is discussed in 7.5.

7.4.2 Horizontal Strains

The effect of traffic action on horizontal average strains is shown in Figure 7.10(a), (b), (c), (d) using the normalised form of average strain. The general trend is for a decrease in both longitudinal and transverse strain under the action of traffic. This is indicative of the stiffening effect on the flexural stiffness of the total pavement structure that is being caused by the compaction of the surface course. It is interesting to note how significant this is when one considers that the surface course is a relatively thin layer of relatively soft asphalt concrete overlying a much

These pages are fold-out sheets of Figures 7.7 to 7.18.

For binding purposes they have been placed at the end of this chapter after p.203.

7.4.2 (cont.)

thicker base of rigid portland cement concrete. Its role is so significant because the surface course is the outer compression fibre of the whole pavement. This evidence of flexural stiffening is corroborated by the comparative effect of temperature. The low temperature, $25^{\circ}C$, produces smaller horizontal strains than the high temperature, $40^{\circ}C$, because the mix is stiffer at lower temperatures.

7.5 THE INFLUENCE OF PAVEMENT THICKNESS

7.5.1 Apparent Dynamic Modulus

It has been demonstrated that the apparent dynamic modulus is not solely a characteristic of the material. It also appears to be a function of the surface course thickness. Figure 7.9 shows how the modulus varies with air voids content for each strip, i.e. for different thicknesses. This data is re-presented in Figure 7.11(a), (b), to show how the modulus varies with surface course thickness for various air voids contents. The curves in ib. (a) show a very marked increase in modulus as the layer thickness decreases. This is apparently the result of the confining effect had by the frictional stresses at the layer boundaries. A similar trend in ib. (b) is upset only by low modulus values for the 20 mm thick pavement. Little weight is given to those low values however because the gauge had a very short gauge length and provided a number of problems in calibration.

The greater confining effect in thin layers implies that for given conditions they will be more stable and better able to resist deformation than thicker layers. These results confirm the postulates and evidence of McLeod[39] and Goetz et al[42] mentioned in Chapter Two. However, the steepness of these curves is surprising especially in view of the relatively small influence of layer thickness on densification that was demonstrated in the previous chapter. Are there some extraneous factors influencing these results? This question can be answered in two ways. One is to progressively eliminate various factors such as mensuration technique, etc., and the other is to compare the results with the predictions of a theoretical analysis.

So far as mensuration technique is concerned, most sources

7.5.1 (cont.)

of error have already been examined either earlier in this chapter or in the author's publication, op. cit. One remaining matter concerns confinement of the material by the coils themselves. This has been recently studied by Morgan[116] using sandy soils. He found that a certain amount of confinement occurred with rough-faced coils causing under-estimation of strain and that smooth-faced coils tended to prevent compaction causing an over-estimation of strain. However both effects were only significant at strains greater than 1.5% which is considerably greater than the strains being measured here. The implication for the present case of use in asphalt concrete is a matter of conjecture. The particle size is much larger than the sand under study by Morgan and the coil discs are physically bonded to the material. These two factors may imply that the "rough-faced coil" situation of Morgan's study applies and this would result in lower strains over shorter gauge lengths and hence a higher apparent dynamic modulus. The author feels that this is unlikely to be true however because (i) the mix is relatively stiff, (ii) the particles are not moving significantly and (iii) the coil disc is probably acting in a similar manner to a large flaky particle.

Analysis of the situation by three-layer elastic theory gives the relationships shown in Figure 7.12. The assumed elastic modulus values are 6000 and 200 MN/m^2 for temperatures of 25^oC and 40^oC respectively, i.e. the same as previously used. The change in average vertical strain, presented in inverse form as the apparent dynamic modulus, is very slight as the surface course thickness changes. The slight increase in modulus that is evident as the layer thickness decreases is probably due almost entirely to the increasing flexibility of the structure and not to any confining effect. When these curves are compared with those measured at the Testing Track a great difference is noticeable. Disregarding the actual magnitude of the elastic analysis values which are dependent on the chosen elastic modulus, it is obvious that the Track measurements are displaying a highly significant confining effect that is not shown by the elastic analysis. The probable reason for this

7.5.1 (cont.)

is as follows. One of the fundamental assumptions of the elastic analysis is that the layers are composed of homogeneous isotropic materials. While this assumption may be acceptable on the large scale when considering the whole pavement structure, it is certainly not valid when considering one layer in detail especially if the material is far from being either homogeneous or isotropic. Reference to Figure 7.13 will show that the large size of the particles in relation to the layer thickness enables them to cause significant arching effects and to develop a great deal of resistance to deformation. This is the reason why the apparent dynamic modulus rises so steeply as the layer thickness is reduced towards twice the maximum aggregate size.

With this mechanism it is now possible to explain the paradox of densification being little affected by layer thickness. The imposed stresses are still able to cause some change in the closeness of packing of the aggregate particles through flow in the binder film although the particles are constrained from making any movements of relocation by the boundary frictional stresses and the relative size of the particles themselves. Hence the process is essentially one of densification rather than one of compaction with associated structural changes. This supposition is confirmed by reference to Figure 6.13 which shows that the increase in stability, which is one measure of restructuring, of the thin Strip 28 is very small for its associated increase in density.

The conclusions from this section are significant. Thinner surface course layers, especially those little more than double the maximum aggregate size in thickness, display a very much stiffer response to a moving wheel load than do thicker layers. This is primarily the effect of arching between aggregate particles constrained from moving by their large size in comparison to the layer thickness and by boundary frictional stresses. This stiffer response however does not hinder densification but it does inhibit restructuring of the pavement material.

7.5.2 Horizontal Strains

Both longitudinal and transverse strains increase as the surface course thickness increases, Figure 7.10. This implies that the measuring location, the mid-depth of the surface course, is at an increasing height above the neutral axis as the thickness increases and hence the neutral axis must remain fairly stationary in position. Comparison of these measured values with those predicted by the elastic analysis is made in Figure 7.14. Great disparity is evident. The measured "strains" are twenty times larger than the calculated strains, as discussed previously. They also increase significantly as thickness increases whereas the calculated strains decrease. This could happen if the neutral axis of the pavement were lower than predicted by elastic theory and if it remained fairly stationary. No other satisfactory explanation can be offered.

7.6 DEPENDENCE ON TEMPERATURE

7.6.1 Vertical Strains

An increase in temperature softens the mix and causes a considerable increase in vertical deformation. This is demonstrated by the decrease in apparent dynamic modulus with increasing temperature for the final stable state shown in Figure 7.15. An increase in temperature from 20°C to 40°C causes a 25-fold increase in vertical average strain or decrease in apparent dynamic modulus. Unfortunately many of the 40°C measurements during the testing program were not obtained for various non-technical reasons and the trend of these values with traffic and layer thickness were not graphed. However, the basic trend is the same as at the low temperature although the changes are very much smaller. This is to be expected since packing and confining effects are unlikely to be significant when the binder viscosity is low and provides little constraint to the aggregate structure.

7.6.2 Horizontal Strains

Horizontal average strains also increase as the temperature increases, Figure 7.10 cf (a), (b) and (c), (d). This is the result of the greater deflection of the pavement due to the decreased stiffness of the compressive layer and also of the lower

7.6.2 (cont.)

modulus of the material itself which gives rise to higher strains for similar stresses. Comparison with the elastic analysis in Figure 7.14 again shows a significant disparity that is inexplicable.

7.7 THE INFLUENCE OF FOUNDATION SUPPORT

The influence of foundation support on the dynamic response to a moving wheel load may be assessed by comparing the two sections of the Motorway Test Area. Figure 7.16 shows the vertical response in terms of the apparent dynamic modulus as a function of air void content. A large amount of scatter is evident and the curves for the two cases have different shapes. In particular the curve for the granular base seems abnormal. Such a steep rise for such a small increase in density is highly unlikely, of Figure 7.9, and it is probable that the low initial values were the result of gauge seating problems. The base coil was difficult to secure to the granular base but construction compaction ought to have overcome any looseness of seating. For ease of placement of the surface coil the mastic was softened by the addition of some oil and the coil must have required trafficking to seat it correctly.

After 8 months of traffic the vertical average strains are generally smaller for the granular base than for the concrete base, i.e. the apparent dynamic modulus is higher. This is expected from the predictions of elastic theory which showed, section 3.2, that vertical deflections and horizontal strains would be higher and vertical strains would be lower for a "flexible" base than for a "rigid" base. The difference is very small here but then that is not surprising when the supposedly "flexible" granular base is high quality motorway construction.

Horizontal average strains, Table VI-V have decreased under traffic action and they are higher for the granular base than for the concrete base.

7.8 PERMANENT STRAINS

The strain gauges also provided a coarse measure of permanent average strain from the amplitude of the null balance signal which had been calibrated against gauge length. The accuracy of this

7.8 (cont.)

measurement was to the order of $\pm$ 400 x 10^{-6} a.s. which incorporates the degree of resolution and the expected variations in the dynamic instrument which was not designed for long term stability.

Significant permanent average strains of the order of 20 000 to 40 000 x 10^{-6} a.s. were measured and these are presented in Figure 7.17. The permanent strain increases under the action of traffic and the biggest increases occur during the high temperature stages. The magnitude of these strains appear to have little correlation with thickness but if these are compared with the associated changes in density as indicated by air voids content there is a fair correlation, Figure 7.18. This correlation with density change is very much better than that found in the previous chapter for permanent deformations measured by the less precise technique of the profilometer, cf. 6.2.5.

The anomalous decreases in permanent strain that may be noticed in Figure 7.17 occur for the two strips, 22 and 26, where heating failure caused local overheating and rutting distortion.

From the permanent strains a rough estimate can be made of the residual average strain per passage of a wheel load. On an average over 125,000 vehicle passes this is 0.2 to 0.3 x 10^{-6} a.s. per pass and hence it is likely to have ranged from approximately 1 x 10^{-6} a.s. per pass at the beginning of trafficking to zero at the end. Such small strains were beyond the range of the individual dynamic measurements and so it is not surprising they were not detected at that stage.

7.9 CONCLUSIONS

(i) Measurements of the transient strains occurring under a moving wheel load have provided valuable information on the dynamic response of asphalt concrete. The measured profiles of transient average strain show characteristic shapes that are similar to those quoted in other literature, although these are the first known direct measurements of transient vertical strain.
The profile of vertical strain was similar in magnitude to what was expected although the extension occurring immediately before

7.9 (cont.)

contact was sometimes very high even to the extent of exceeding the succeeding compression. Such significant extensions appeared to be the result of complications in instrumentation, q.v. (vi) and not an indication of true behaviour.

Horizontal average strains were compressive and were higher by one order of magnitude than those predicted by elastic theory but the profiles were of the expected simple shape. A small part of this apparent error was due to complications in instrumentation similar to those above and part may have been due to the high horizontal shear stresses over the tyre contact area.

(ii) Peak vertical average strain decreased as the pavement was compacted by traffic. Presented in inverse form as a ratio of imposed contact pressure these peaks were converted to an introduced term "the apparent dynamic modulus" which was a measure of the dynamic stiffness in the vertical direction. This modulus, then, increased under traffic action. It was also greatly increased by a decrease in surface course thickness by considerably more than could be attributed to an increase in flexibility. The postulated cause of this strong dependence on layer thickness was the magnitude of the ratio of layer thickness to maximum aggregate size, i.e. the relative dimensional effects. The modulus increases steeply as the ratio decreases towards 2 which is the lower limit of practicability. An increased dynamic stiffness caused by the thinness of a layer does not however prevent densification although it does inhibit internal restructuring.

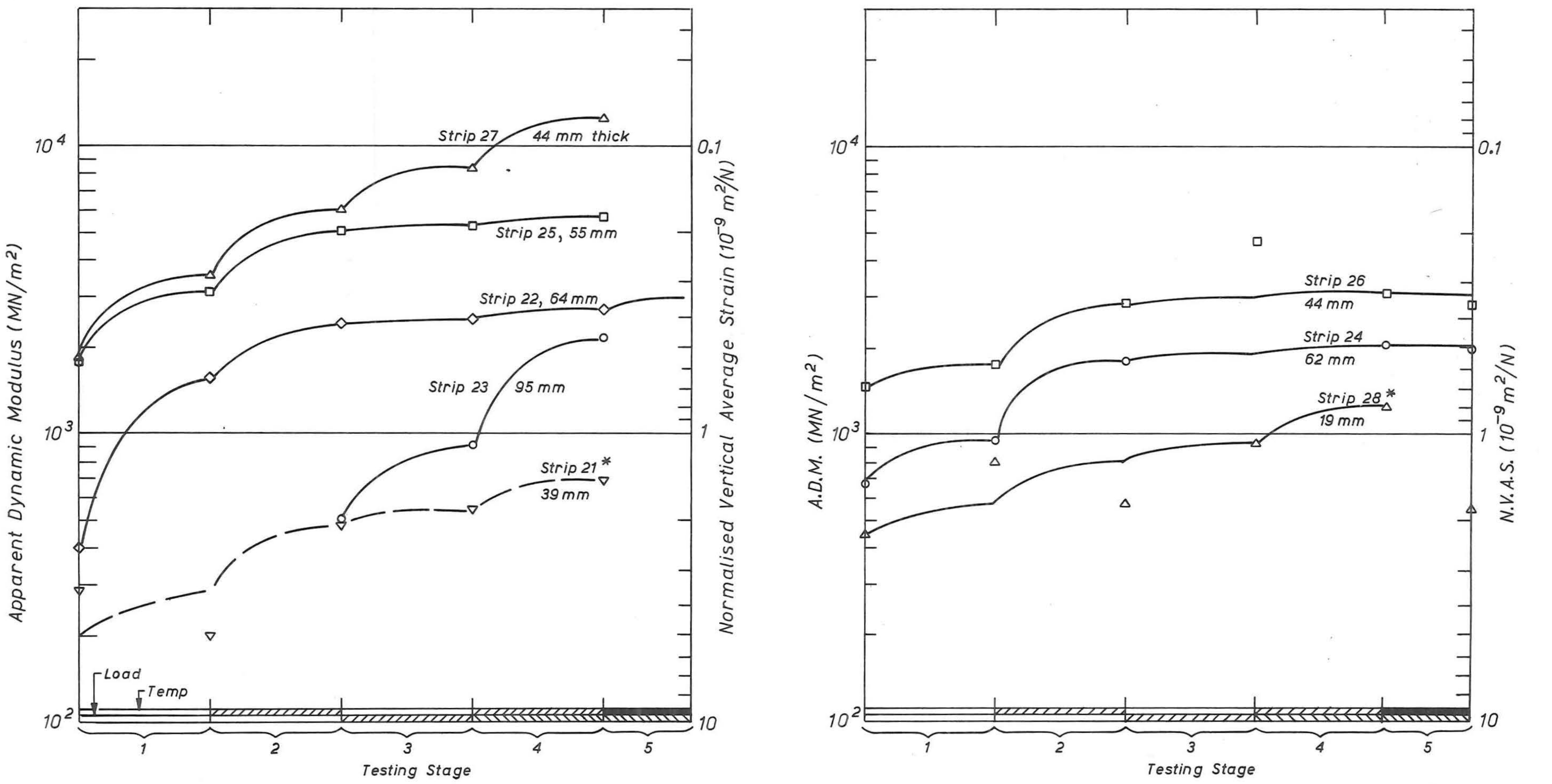
(iii) Vertical strains and the apparent dynamic modulus are highly sensitive to temperature. The effect of temperature on horizontal strains is dependent on the flexibility of the underlying structure - for the concrete base in the Testing Track study there was little change.

(iv) Foundation flexibility has a slight effect on measured vertical average strains but a greater effect on horizontal average strains.

(v) Permanent deformations expressed on a unit thickness basis were closely similar to the associated density changes but not to layer thickness.

7.9 (cont.)

(vi) The induction coil strain gauges used in this study performed well although they were being used near the limit of their sensitivity at low pavement temperatures and this caused a reduction in resolution. The proximity of metal in the moving vehicles was marginal and caused significant complications for gauge lengths greater than 1.8 times the coil diameter. This factor imposes some limitations on the use of this technique for surface course observations.



(a) Mix B (Temperature 25°C)

(b) Mix F (Temperature 25°C)

Fig. 7.7 APPARENT DYNAMIC MODULUS AS A FUNCTION OF TRAFFIC TESTING CONDITIONS

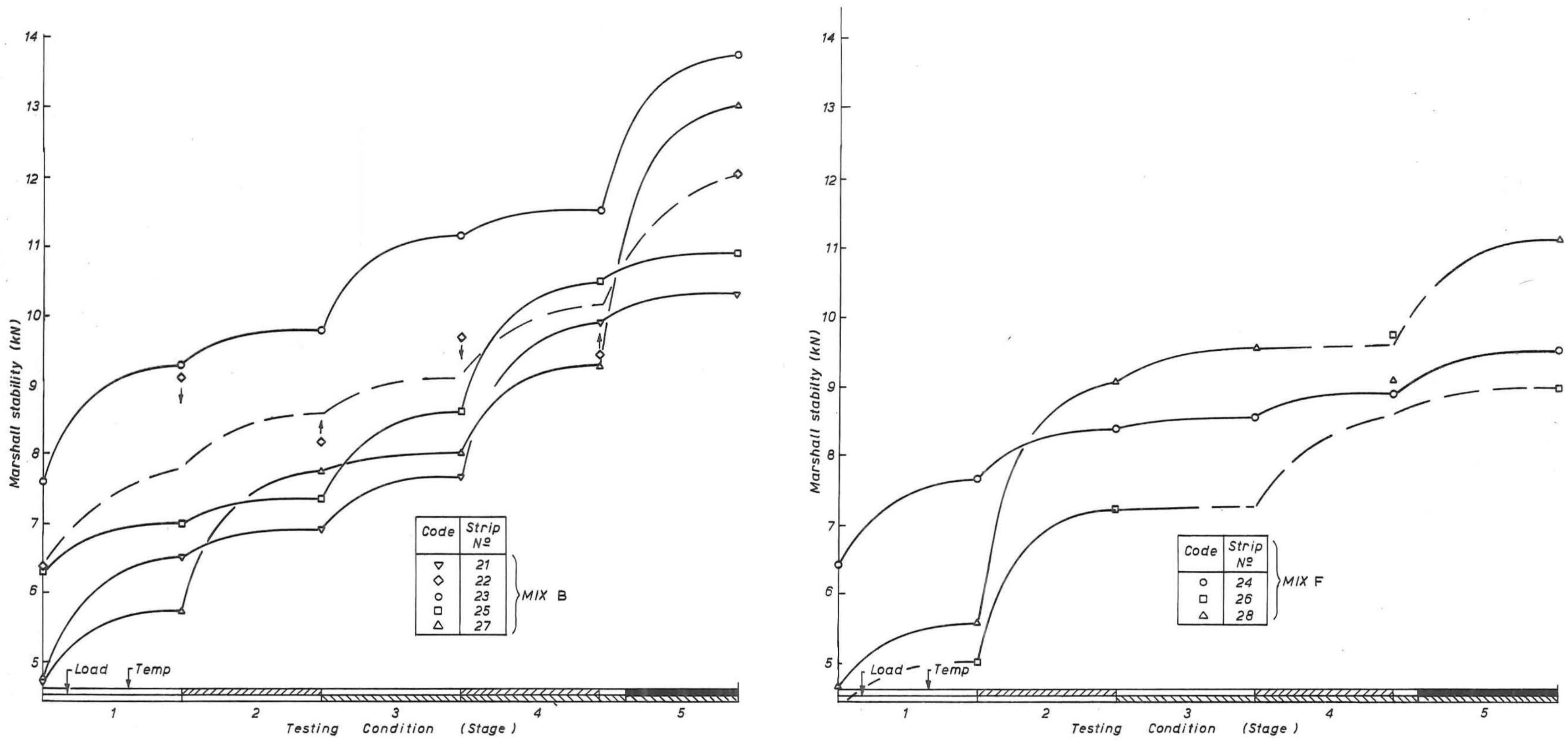
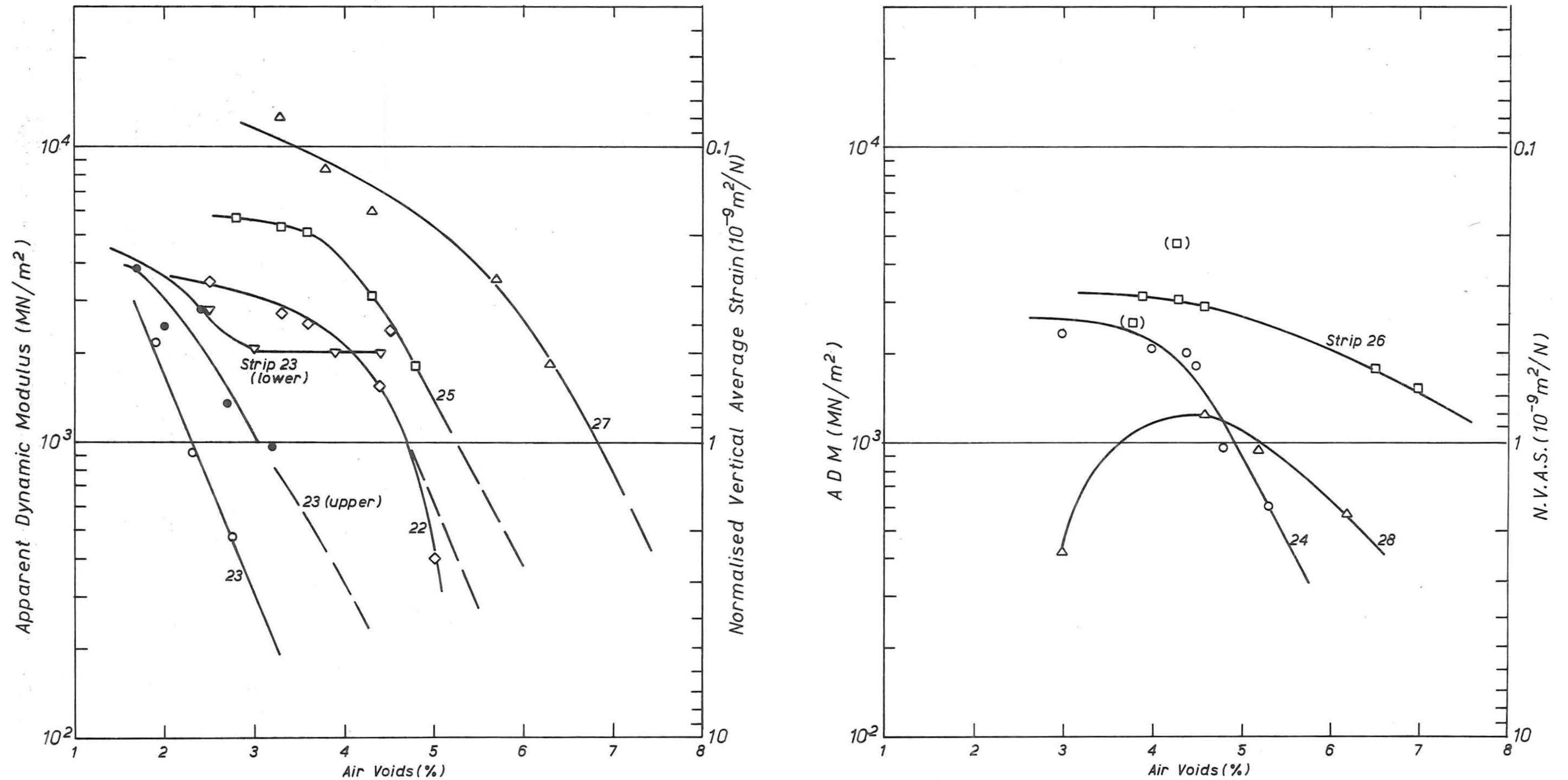


Fig. 7.8 MARSHALL CORE STABILITY & TESTING CONDITIONS



(a) Mix B (Temperature 25°C)

(b) Mix F (Temperature 25°C)

Fig. 7.9 APPARENT DYNAMIC MODULUS AS A FUNCTION OF AIR VOIDS

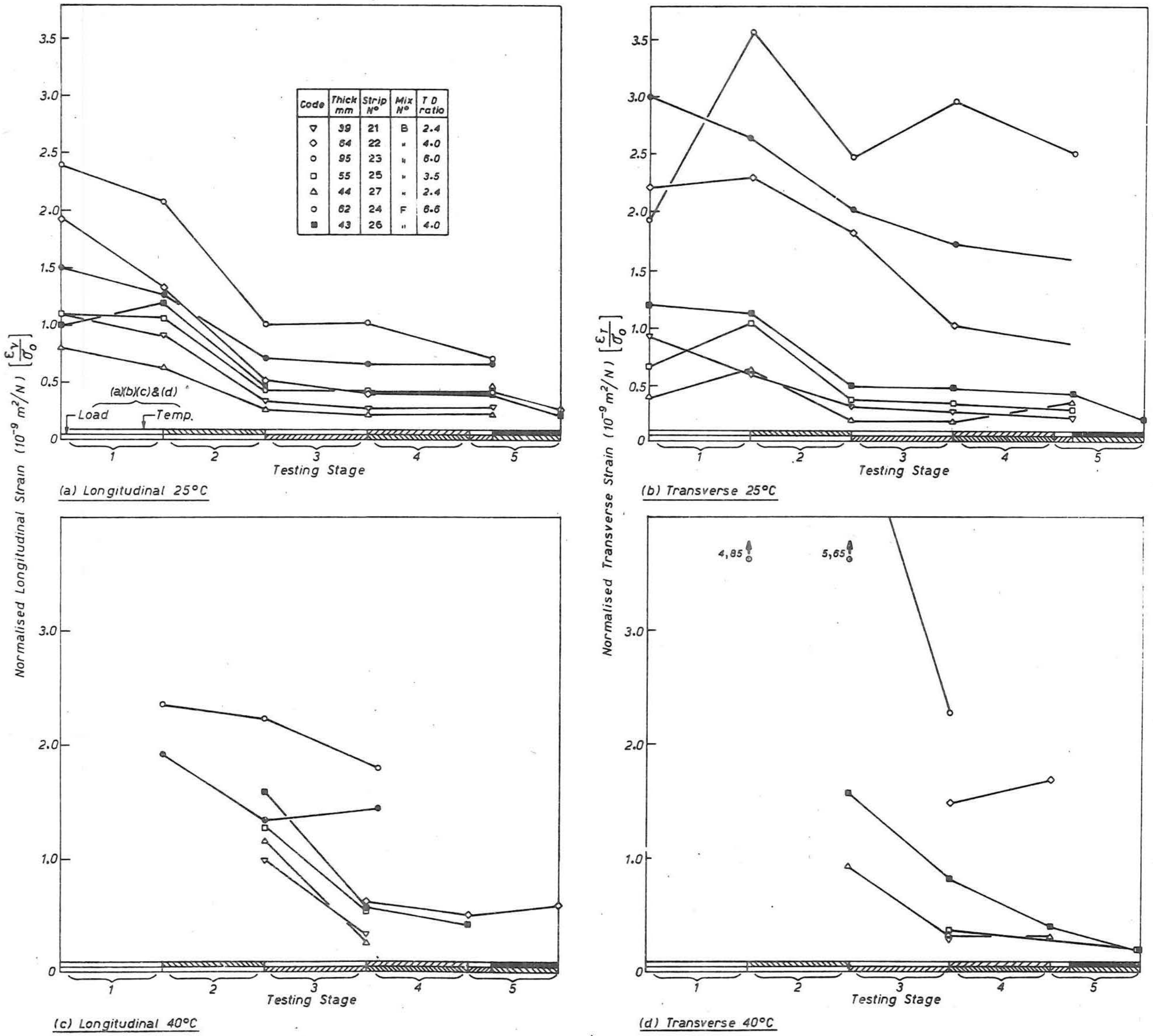


Fig. 7.10 HORIZONTAL AVERAGE STRAINS AT MID DEPTH, TESTING TRACK

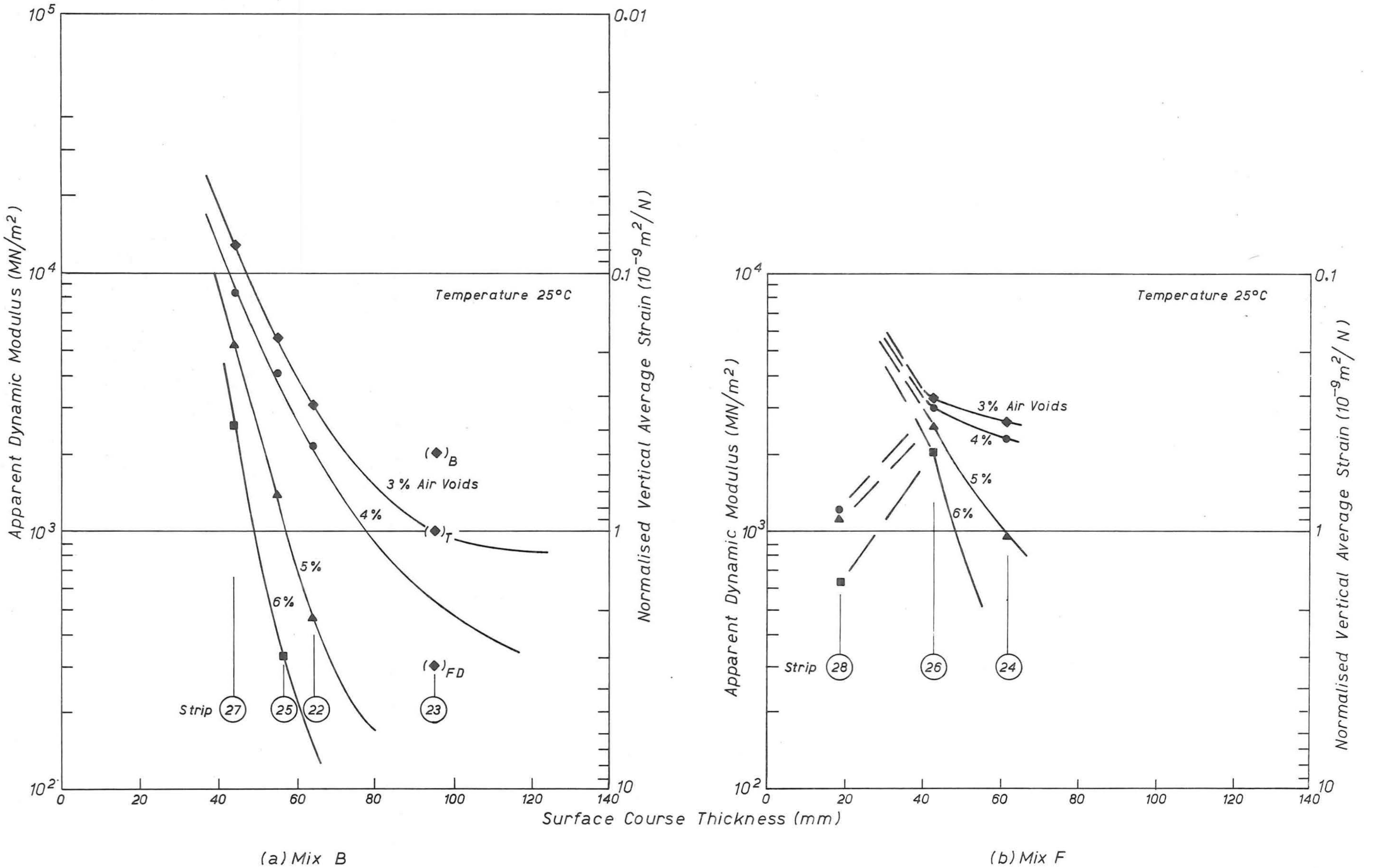


Fig. 7.11 APPARENT DYNAMIC MODULUS AS A FUNCTION OF SURFACE COURSE THICKNESS

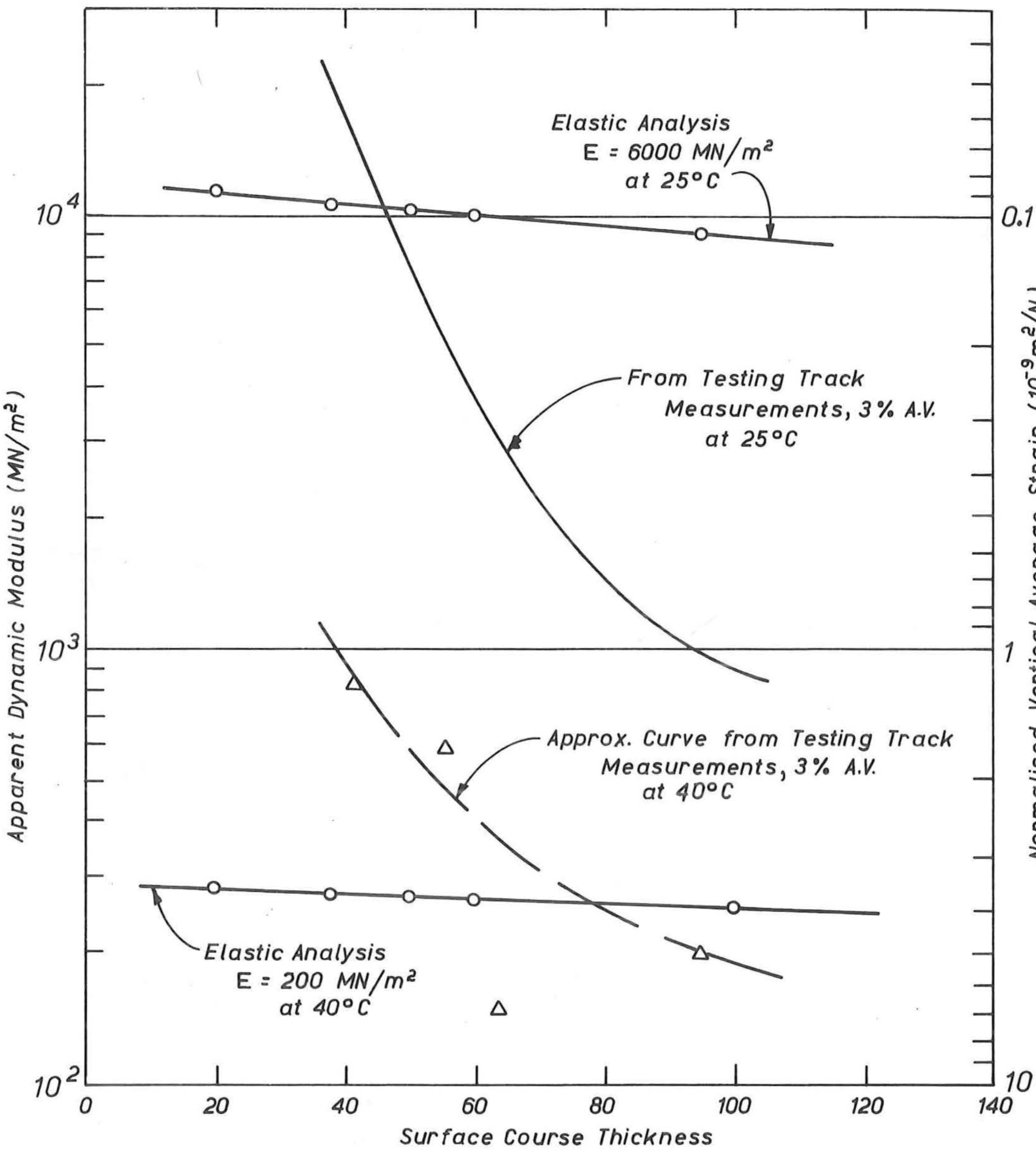


Fig. 7.12 COMPARISON OF MEASURED APPARENT DYNAMIC MODULUS WITH ELASTIC ANALYSIS PREDICTIONS

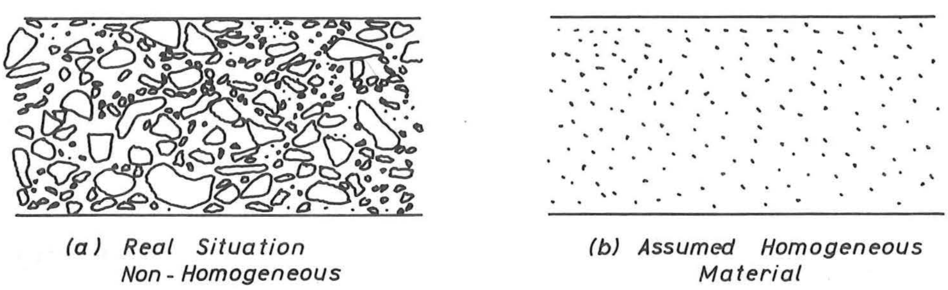


Fig 7.13 RELATIONSHIP OF PARTICLE SIZE AND THICKNESS

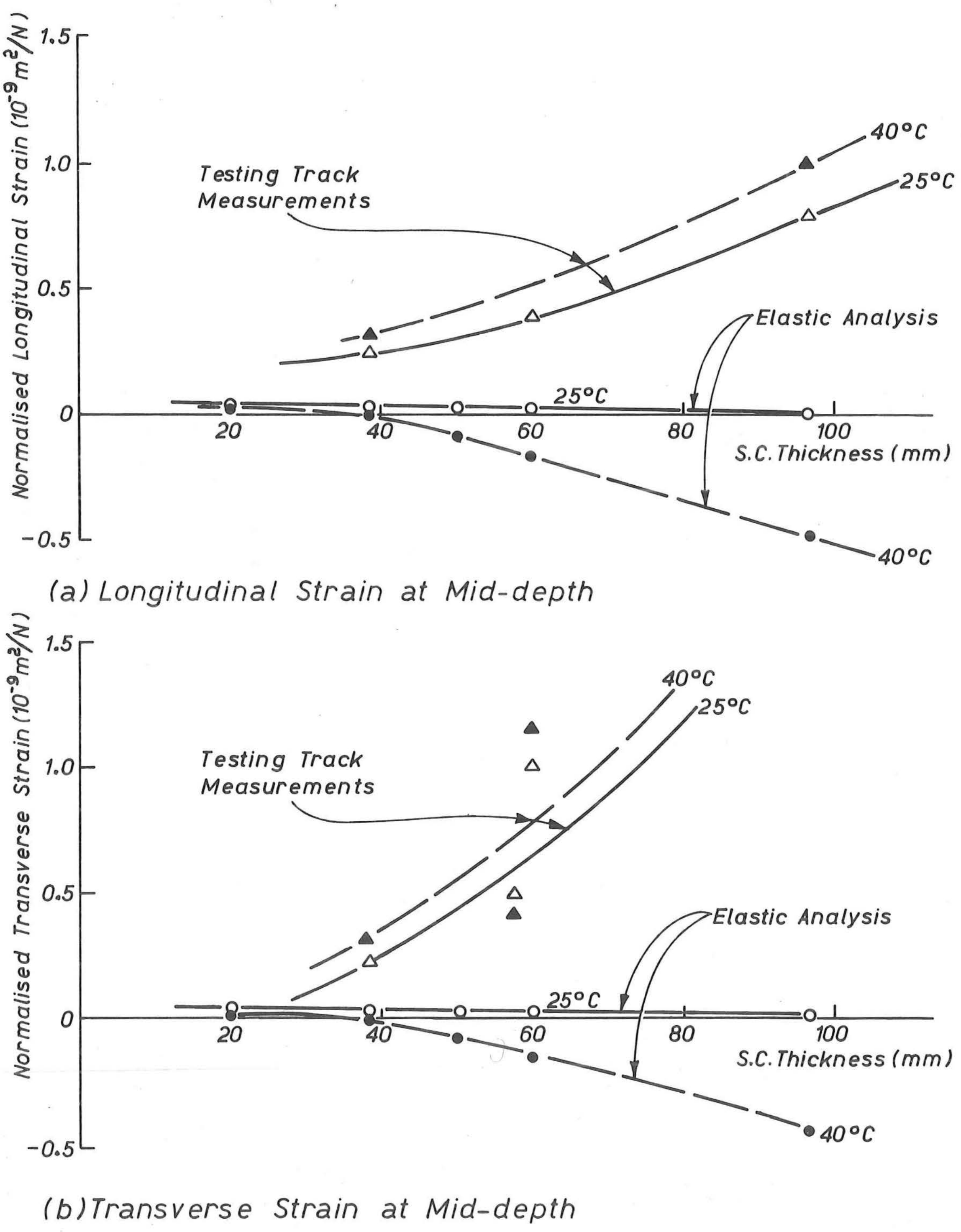


Fig. 7.14 COMPARISON OF MEASURED HORIZONTAL STRAINS WITH ELASTIC THEORY

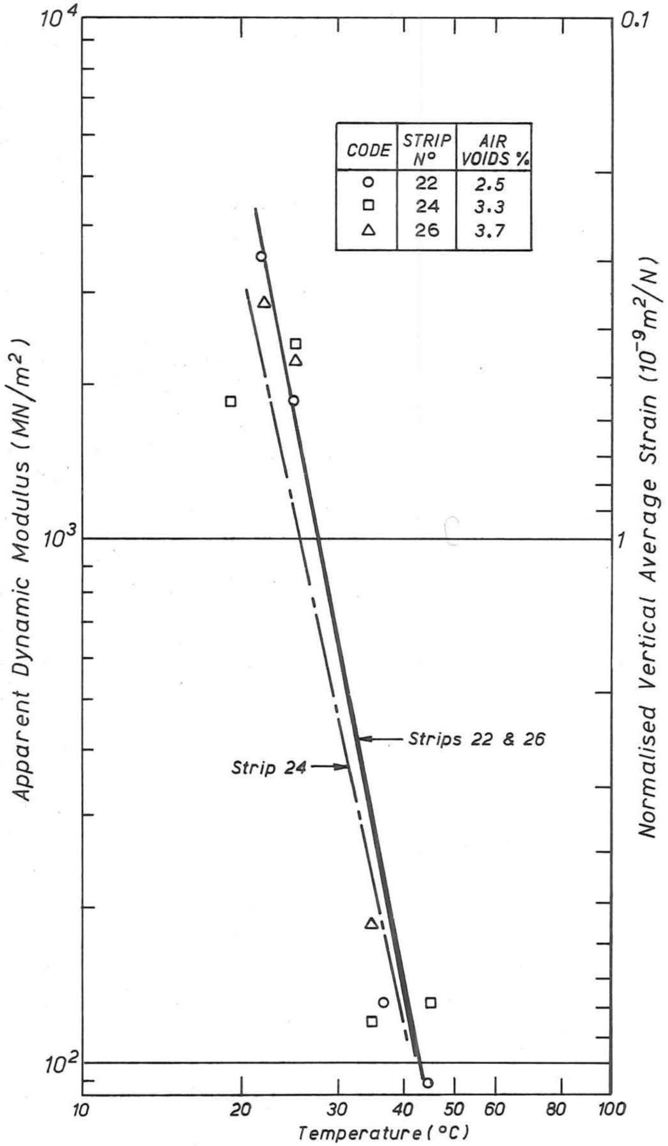


Fig. 7.15 THE EFFECT OF TEMPERATURE ON THE APPARENT DYNAMIC MODULUS, TRACK TEST SERIES 2, STAGE 5

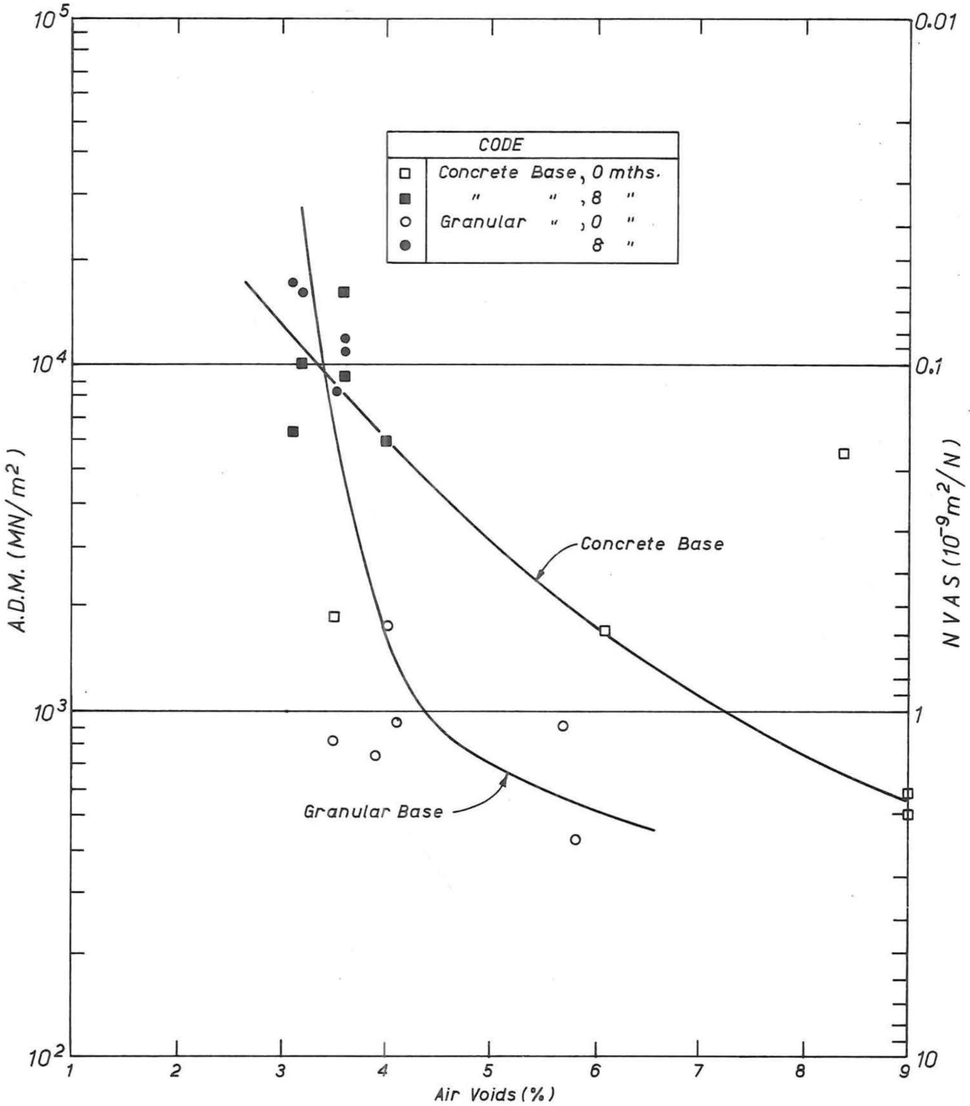


Fig. 7.16 THE EFFECT OF FOUNDATION FLEXIBILITY ON THE APPARENT DYNAMIC MODULUS - NORTHERN MOTORWAY.

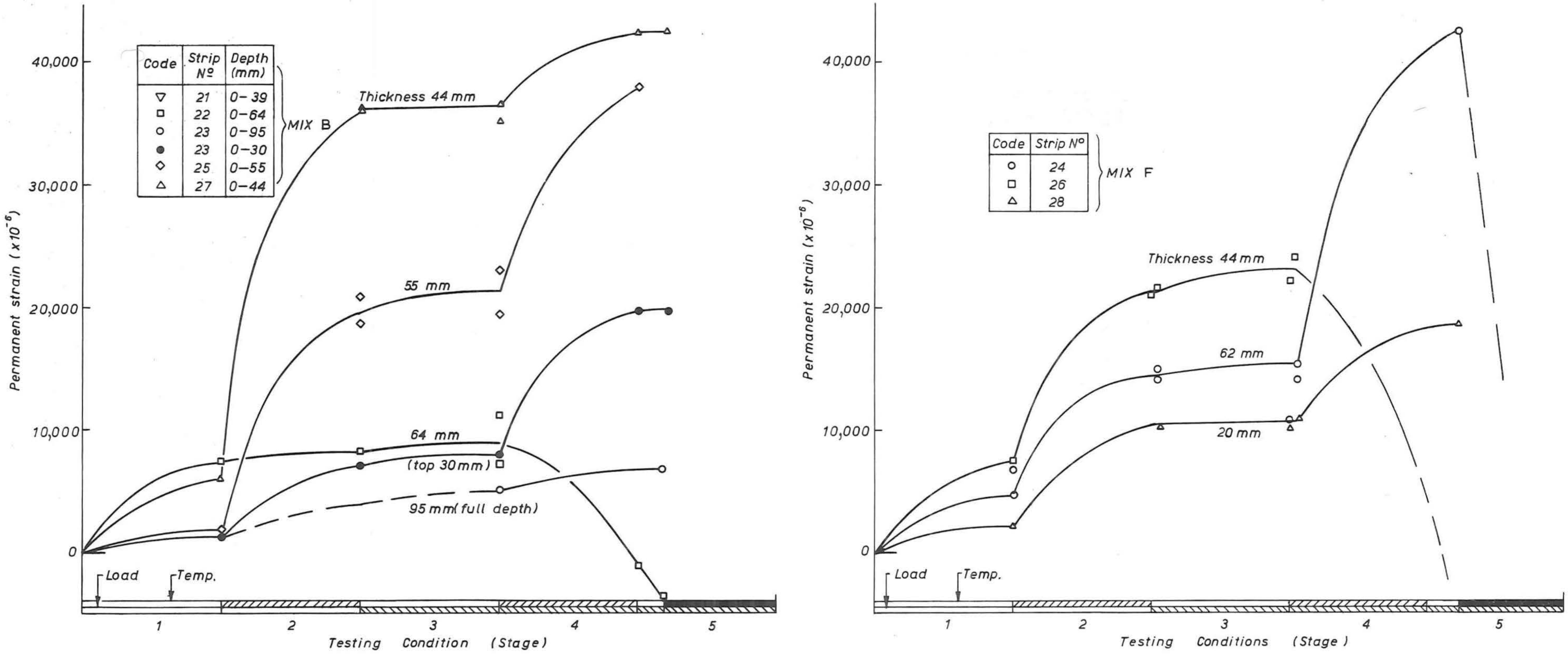


Fig. 7.17 PERMANENT VERTICAL STRAINS

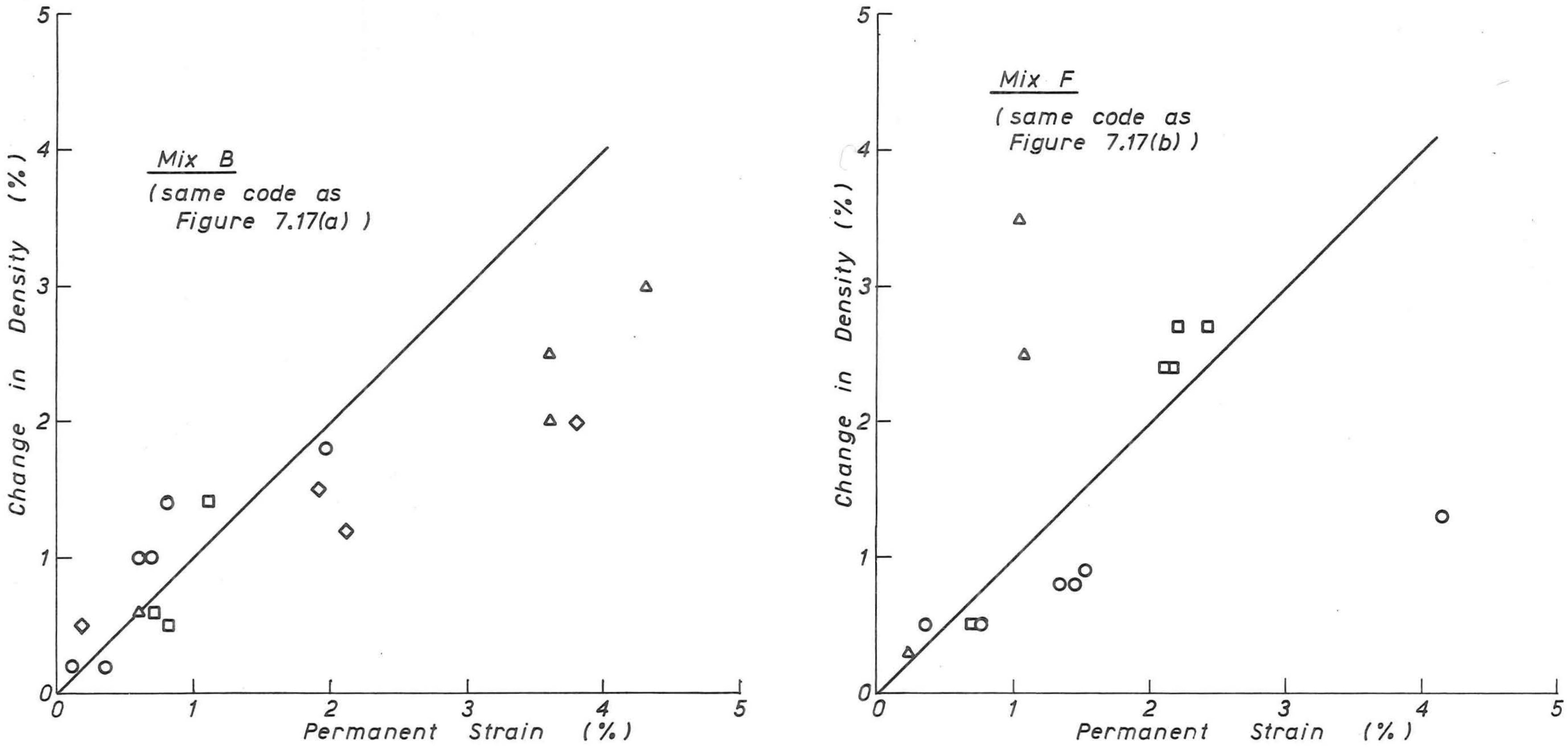


Figure 7.18 COMPARISON OF CHANGE IN DENSITY AND PERMANENT STRAIN

CHAPTER EIGHT

CONCLUSIONS AND IMPLICATIONS

The principal conclusions drawn from this study are summarised and a thesis is presented on the mechanism of traffic compaction of an asphalt concrete surface course. Finally, the implications of these findings are considered both for international knowledge and for practice in New Zealand.

8.1 CONCLUSIONS

The aim of the study was to determine the principal factors involved in the phenomenon of compaction of asphalt concrete surface courses by vehicular traffic during normal use of the pavement. Consideration was given to the material behaviour of asphalt concrete and to the role of the surface course in the pavement structure and the conditions surrounding it. This led to the choice of a fullscale pavement testing track as the mode of testing because such a facility provides controlled conditions. Comparative studies were also conducted at two sites on normally trafficked highways. The principal conclusions from the whole study are summarised below.

8.1.1 The stresses existing under the wheels of vehicular traffic are of sufficient magnitude and are of suitable direction to cause compaction of dense graded asphalt concrete surface courses under normal environmental conditions (cf. 1.2 and Chapters 5, 6, 7). The extent to which such compaction occurs is dependent on a number of factors covered below.

8.1.2 The process of compaction is distinguished from densification. Densification is simply defined as the process which causes an increase in unit mass of a material. Compaction generally results in densification but it also includes restructuring of the relative location and orientation of particles within the material. (cf. 1.2) Evidence of this restructuring was obtained. (cf. 6.8)

8.1.3 Under the action of traffic, the rate of compaction is initially high but decreases over a period to zero. Up to 50% of the total change in compaction indicator occurs during the first 100,000 to 200,000 passes of mixed vehicular traffic on a normal highway or in the first 500 to 1000 passes of the Vehicle on the Pavement Testing Track. The actual rate of compaction depends on the imposed conditions and the initial state of the material. The scatter of indicator values generally decreases under the action of traffic. (cf. 6.3)

8.1.4 Asphalt concrete is defined by the author to be in a "stable state" for certain imposed conditions when it is resistant to further compaction by traffic, i.e. when the compaction rate has decreased to zero. Such a stable state is reached after relatively few thousand passes if the initial state is close to the stable state. However if the

8.1.4 (cont.)

change from initial to stable state is large then the rate of compaction decreases more slowly and the stable state will only be reached after approximately 10,000 passes of the Testing Track Vehicle or two million vehicles of normal highway traffic. (cf. 5.1, 6.3)

8.1.5 The magnitude and nature of stable state conditions for given initial conditions depend on the maximum imposed contact pressure, the maximum duration of a loading cycle and the minimum binder viscosity, i.e. maximum pavement temperature. On the Testing Track all these factors were controlled and a semilogarithmic function was derived relating the stable state air void content to imposed contact pressure and binder viscosity conditions. The function demonstrates how closely dependent the stable state is on loading and binder viscosity. The stable state densities obtained on the Testing Track under high contact pressure and high temperature conditions were generally close to Marshall predictions. The important finding is that design conditions can always be exceeded if the imposed conditions are sufficiently severe. (cf. 6.3, 6.6)

8.1.6 Mix design is a determinative factor of the stable state achieved by traffic compaction. For example, differences in design bulk density, or alternatively air voids content, are preserved under the action of traffic. This was particularly apparent when the binder content was varied for a constant aggregate gradation and it highlights the important role of the binder film thickness. (cf. 6.6)

8.1.7 Density and air void content were only good indicators of the resistive state of the mix when related to design values and initial or construction values. They were not definitive indicators of a stable state because they did not depend on imposed conditions alone. Marshall stability of extracted core samples certainly increased under trafficking but it was unrelated to imposed conditions principally due to the anisotropic properties of the cores. Bearing capacity, as calculated from Marshall values of extracted cores, was also not directly related to imposed conditions. Hence none of these indicators are definitive of the stable state developed under certain imposed conditions. No other core properties were tested as indicators. (cf. 6.2)

8.1.8 The transient vertical response of asphalt concrete to a moving wheel load displays a well-defined and characteristic profile. A small extension occurs immediately prior to contact by the tyre followed by a steep rate of compression which lags the stress pulse considerably. The peak compression therefore occurs just as the tyre ceases contact. It is followed by a small amount of rapid decompression and a much slower rate of decompression lasting up to 5 or 10 seconds. Such a profile illustrates the viscoelastic nature of the material and it is distinctly different from the symmetrical profile for an elastic material.

The maximum residual permanent strain per vehicle pass for the Testing Track was approximately 1×10^{-6} average strain and it would be less under the higher speeds of highway traffic.

An introduced unidirectional measure of the vertical dynamic response termed the apparent dynamic modulus was defined as the ratio of contact pressure to peak vertical average strain. It increased during trafficking demonstrating an increased stiffness but it was not definitive of stable state conditions because it depended on layer thickness and mix design. (cf. 7.3 to 7.9).

8.1.9 The dynamic response in terms of horizontal strains also displayed well-defined characteristic single peak profiles and these compared well with published data. The time base corresponded to approximately double the length of the tyre contact area and the peak values were generally compressions of the order of 200 to 300×10^{-6} strain. These are much greater than predictions by elastic theory and seem unusual by being of the same order as associated vertical strains. This is still the case after due allowance had been made for a complication in instrumentation. One contributing cause may be horizontal shear stresses over the tyre contact area. (cf. 7.3 to 7.9)

8.1.10 Resistance to compaction in the mix is largely developed in the binder film. Interparticle contact plays a major part in a load-bearing capacity but adjustment of particle structure within the mix is determined by the viscous resistance developed in the binder film. Hence a thin film and a high viscosity develop high resistance to compaction. (cf. 2.2, 2.3, 6.9.5)

8.1.11 Initial conditions such as construction density play a

8.1.11 (cont.)
significant role in compaction characteristics. A low initial density produces a low stable state density because the interparticle structure has not been moulded into its most stable position. Only under imposed conditions of heavy loads and high temperatures, or low binder viscosities, will the stable state reach the same level as that for an initially well compacted mix. (cf. 6.4)

8.1.12 The thickness of the surface course has little effect on densification but it does influence the restructuring component of compaction. A thin surface course undergoes generally slightly less densification than a thick surface course due to the influence of boundary frictional stresses. However a thin surface course exhibits a very much stiffer dynamic response than a thick course due to the relatively large dimensional ratio of aggregate particle size to layer thickness. This stiffer dynamic response of thin layers tends to inhibit movement of the aggregate particles thus inhibiting the re-structuring component of compaction and also stabilising the layer against excessive distortion. The proneness of thick layers to distortion is principally a result of the low confining effect of boundary stresses. 'Thin' and 'thick' as used here are relative terms implying ratios of thickness to maximum aggregate size of approximately 2 to 3 and greater than 5 respectively. (cf. 6.5, 7.5)

8.1.13 The effect of compaction diminishes with depth in the surface course mainly as the result of the negative temperature gradient but also partly due to the decreasing influence of surface horizontal stresses and compressive flexural stresses. (cf. 6.5)

8.1.14 The flexibility of the underlying pavement structure has little influence per se on the densification of the surface course. What it does influence however are the horizontal or flexural compressive stresses and strains which help determine the stable orientation of particles. These strains may also cause tensile failure of the binder film resulting in a surface texture with fine longitudinal surface cracking and the possibility of flushing at low air void contents. (cf. 6.7)

8.1.15 The occurrence of permanent deformations in the surface course

8.1.15 (cont.)
during traffic compaction is mainly due to the associated increase of density. However, resistance against distortion, such as by rutting, is mainly a function of the viscous resistance developed in the binder film. For example, even high stability mixes may be badly distorted under certain conditions if the imposed conditions are severe. A thick pavement at high density may distort under heavy loads, high temperatures and slow vehicle speed. Thinner pavements are usually much more stable but may suffer distortion if the binder film is too thick, if the binder viscosity is too low or if inter-particle friction is low. (cf. 6.2 to 6.6)

8.1.16 A stable mix structure in the field characteristically has a predominance of particles aligned in a near horizontal orientation. This particular structure is developed by the characteristic stresses which exist under both pneumatic-tyred construction rollers and under normal vehicular traffic. Such a preferentially oriented structure is anisotropic in behaviour, exhibiting high strength in planes perpendicular to the preferred orientation and low strength in the plane of preferred orientation. (cf. 6.8)

8.2 THESIS ON THE MECHANISM OF COMPACTION OF ASPHALT CONCRETE

When an asphalt concrete mixture is laid by a paving machine it consists essentially of a number of irregularly-shaped solid particles randomly distributed and oriented within a viscous fluid matrix. Construction compaction forces these aggregate particles into a more closely-packed structure. The highly regular nature of the repeated compactive forces together with the constriction of the upper and lower layer boundaries of the surface course cause these particles to be preferentially oriented in a near horizontal position. (Constriction by the upper boundary, i.e. the surface, occurs under the tyre by the action of horizontal shear and frictional stresses existing over the contact area.) The fluidity of the matrix at low viscosity permits this movement. At this stage, the mix has developed a structure strong enough to carry the compactive load with only little deformation but the low binder viscosity makes it still susceptible to distortion by any greater loading intensity.

At normal pavement temperatures, under the action of normal

8.2 (cont.)

vehicular traffic, the internal mix structure will further adjust to resist the imposed stresses if it is not already sufficiently stable. If construction compaction has been light, the internal structure will not be highly developed and traffic action will have a similar effect in moving the particles closer together and possibly reorienting some of them. In order to move a particle into a new position or orientation the imposed stress must be sufficient to overcome the viscous resistance of the binder film. Hence any particular particle may be considered to possess an energy limit that must be exceeded before it can be moved. The energy limit may be exceeded by increasing the load or the limit itself may be reduced by increasing the temperature and thus reducing the binder viscosity. A stable state is postulated to exist for conditions that do not exceed the minimum energy limit in the mixture.

The mechanism of deformation just outlined places emphasis on the viscous resistance developed in the binder film. Other workers, cf. 2.2.3, have shown that these factors, in terms of triaxial parameters, are directly related to the cohesion of the mixture and that internal friction is largely independent of them. This implies that deformation is a function of cohesion and that load bearing capacity is a function of both cohesion and internal friction. Part of the load bearing capacity is developed through interparticle bearing and friction which are largely independent of temperature, loading rate and history and are thus approximately constant. The remainder of the load bearing capacity is developed through cohesion which is strongly dependent on those loading conditions because it is closely related to the viscosity and film thickness of the binder.

The detailed mechanism by which the mixture responds to the passage of a moving wheel load is a time-dependent phenomenon. The small extension which occurs immediately prior to contact by the tyre is a poisson ratio effect that is automatically recovered as the tyre contact pressure is applied.

The time profile of imposed vertical stress is usually nearly rectangular but the profile of average vertical compressive strain displays a significant time-lag and increases nearly linearly to a peak just before the tyre ceases contact. This strain characteristic reflects the viscoelastic nature of the asphalt binder. A small

8.2 (cont.)

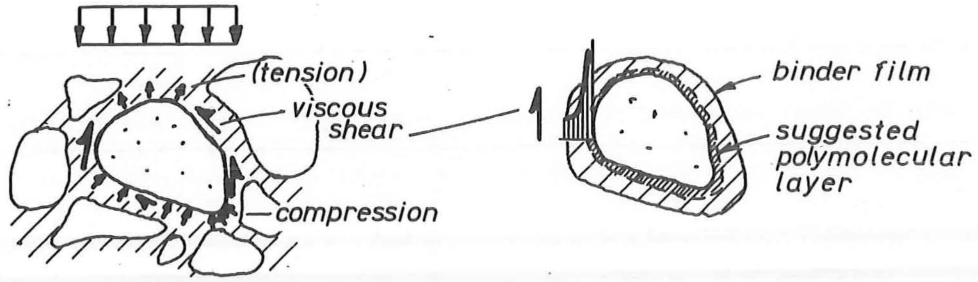
portion of this deformation is elastic and is immediately recovered on unloading. It seems likely that this elastic deformation occurs in a polymolecular layer of the binder firmly bonded to the surface of an aggregate particle, cf. 2.2.5.

The viscoelastic deformation occurs in the remainder of the binder film and is a function of the surface tension which is determined by the film thickness, of the binder viscosity, and of the magnitude and duration of the applied stress. All or most of this viscoelastic deformation is recoverable after unloading but at a slow rate. A diagrammatic representation of the resisting stresses is shown in Figure 8.1(a). Diagram (b) illustrates a boundary layer effect which is a probable cause of the influence of film thickness on effective binder viscosity.

If the viscous shear stresses involved in the viscoelastic deformation were high, some unrecoverable viscous flow will have occurred that results in a residual or permanent deformation. These high shear stresses can occur in the process of reorienting or relocating a particle within the matrix, and they are a function of the same factors that determine viscoelastic deformation. The result of such a restructuring process is a stiffer interparticle structure that will more evenly distribute imposed stresses and thus lessen the magnitude of viscous shear stresses from subsequent load cycles, Figure 8.1 (c), (d). This is another view of the energy limit concept presented earlier.

However, if the viscous shear stresses are excessively high or if the resistance of the binder film is very low then large particle movements may occur and the asphalt may be functioning more as a lubricant than a binder. This causes distortion of the pavement. The extent and nature of the distortion depends on the magnitude and distribution of imposed and resisting stresses.

Repetitive strain cycles tend to work-harden the binder film. This causes an increase in effective binder viscosity and tensile failure may occur in the film under high stresses if it becomes brittle. In high air void content mixes ($>5\%$) oxidation is likely to have a similar embrittlement effect.



(a) Imposed and Resisting Stresses

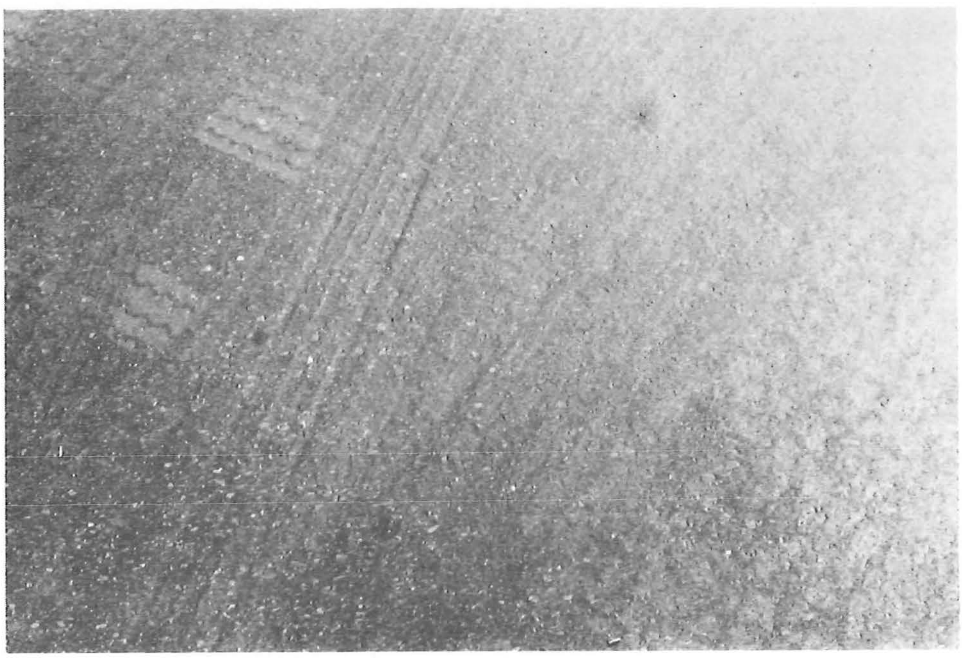
(b) Possible Boundary Layer Effect on Film Resistance

(c) Before Compaction

(d) After Compaction

Author's Mechanism of Stress Relaxation

Figure 8.1 COMPACTION STRESSES AND POSTULATED MECHANISM



Location : Hansen's Lane Intersection,
Blenheim Rd, Christchurch, N.Z.

Binder : 180/200 pen asphalt with 3% oil

Figure 8.2 TYRE IMPRINT IN MIX OF VERY LOW VISCOSITY

8.2 (cont.)

The compaction process, therefore, principally develops a strong and stable interparticle structure which is capable of bearing traffic loads. The extent to which this structure of preferentially oriented particles is developed depends on the developed viscous shear resistance of the binder film which is a function of film thickness and viscosity. When the mixture has developed sufficient resistance to compactive stresses that no further permanent deformations occur, it is said to have achieved a stable state. This postulated mechanism for the behaviour of asphalt concrete precludes any simple concept of an ultimate bearing capacity for the material. Its resistance to imposed stresses has a complex dependence on material and loading parameters, including the history of loading.

8.3 IMPLICATIONS OF FINDINGS

8.3.1 Mix Design and Traffic Compaction

Dense-graded asphalt concrete mixtures of high stability are subject to traffic compaction although the amount that occurs depends on the constructed condition and the imposed conditions of tyre contact pressure, loading duration and pavement temperature. High stability mixes with low deformability or below optimum in asphalt content are highly resistant to compaction but their fatigue strength is probably low. High stability mixes with high deformability or above optimum in asphalt content are prone to excessive compaction and distortion. Neither stability nor deformability ("flow" in the Marshall test) are adequate indicators of resistance to traffic compaction and cognisance is necessary also of the film thickness and effective viscosity of the binder, if suitable measures of these are available.

8.3.2 Mix Design for Incident Conditions

Traffic volume is not the only important factor of the imposed conditions. Tyre contact pressure (as distinct from load), the duration of loading, the duration between consecutive loads, and pavement temperature are all major factors. Hence the rate and magnitude of compaction in this phase is dependent on the nature and volume of traffic and on environmental conditions. Note should be taken of all these factors when considering the effect of traffic

8.3.2 (cont.)

compaction and they would probably account for many of the anomalies apparent in the published studies mentioned in Chapter One. The important design implication of these factors is that where heavy traffic or high temperatures are expected the binder viscosity should be kept high and the film thickness kept low, with due regard to other design considerations, to prevent overcompaction and deformity. Conversely, where light traffic or low temperatures are expected the binder viscosity may be lowered and the film thickness increased if traffic compaction is being relied on to develop full design density in the surface course.

8.3.3 Design Density

The "design" density is not, as commonly termed, the "ultimate" density, it is instead intended to represent the "stable state" density for the imposed conditions of the site under consideration. This stable state density will be exceeded if the imposed conditions become more severe than the expected conditions, and it will not be attained if the conditions are less severe. In actual fact the properties of the design mix as determined by most empirical laboratory procedures bear little relationship to the properties of the compacted mix in the field. This is principally the result of the disparity in interparticle structure between the two situations. For example, the Marshall design procedure produces by impact compaction a randomly oriented wedging structure which is nearly isotropic and construction rolling in the field produces a structure of particles preferentially oriented in a near horizontal position with highly anisotropic properties. This matter of structure is important because it partly explains why the bearing capacity of a mix is not an inherent property but is developed by the stresses imposed on the mix. It also explains why initial conditions can affect the final stable state of the mix.

8.3.4 Compaction Specifications

Specifications requiring construction compaction to achieve 98 to 100% of the design density are generally desirable because they diminish the possibility of large deformations, severe deformity and

8.3.4 (cont.)

oxidation of the binder. These specifications could be relaxed however for thin surface courses since these are generally stable in regard to deformation and tend to compact quickly under traffic. At the other end of the scale, it is virtually impossible to prevent some traffic compaction occurring even if 100% design density is achieved during construction. The amount that occurs however will be very small for high densities. The reason why some traffic compaction will usually occur lies in the slightly different nature of the imposed and resisting stresses during construction and during trafficking. The mix structure will adjust to its stable state for the incident traffic conditions. The only situation where traffic compaction is likely to be non-existent is where the incident conditions are much less severe than the "design" conditions.

8.3.5 Surface Course Thickness

Layer thicknesses between 2 and 5 times the maximum aggregate size will give satisfactory performance for most Marshall-design mixes under New Zealand traffic and pavement temperature conditions. The mixes used in the study for which this applied had stabilities greater than 9 kN (2000 lbs) and flows in the vicinity of 26 0.1mm (10.2 0.01in). Mixes with a low viscosity binder (e.g. $< 1\ m^2/s$ @ 40^oC) or with above optimum asphalt content tend to become unstable at thicknesses in the upper half of the range when being trafficked at high temperatures (c. 40^oC). Mixes with a high viscosity binder (e.g. $> 2\ m^2/s$ @ 40^oC) or with below optimum asphalt content tend to perform quite satisfactorily in thicknesses at the upper limit under trafficking at high temperature.

However thicknesses in the upper half of the range should be treated with caution, especially when traffic is heavy, slow-moving or rigidly channelled and when pavement temperatures are high. They should also be compacted to very near laboratory design density because even small density changes may be accompanied by significant permanent deformations.

Thin layers with a minimum thickness of twice the maximum aggregate size are stable for most common mix designs. Asphalts of very low viscosity however should be used with extreme caution, cf. 8.3.6.

8.3.5 (cont.)

Thin layers tend to densify more slowly under traffic than thick layers although the difference is slight. Variations in thickness cause little variation in density but significant variation in permanent deformation in proportion to the thickness variation.

8.3.6 Use of Low Viscosity Binder

The recent advocacy of very soft thin surface course treatments in New Zealand may be desirable in terms of fatigue design but it appears to be unsatisfactory in terms of its own response to traffic action. The binder viscosity is so low that during warm weather such mixes appear to be unstable and imprinting under vehicle tyres has been observed, Figure 8.2. As a consequence the surface texture becomes smooth and close losing its skid resistant texture. The layer also loses its effectiveness as a good running surface becomes underlying deformities are reflected and some washboarding can occur under the action of braking stresses. If such soft mixes are desired the maximum aggregate size must be close to half the layer thickness in order to achieve some stability under traffic, but even then there is a lower limit on binder viscosity. In the final analysis the principle function of a pavement is to provide a satisfactory running surface.

8.3.7 Resistance to Deceleration Stresses

The anisotropic behaviour of the preferentially aligned mix structure is desirable under normal traffic but its lower strength in the horizontal plane may cause the mix to be unstable against high deceleration stresses, e.g. washboarding sometimes occurs near urban intersections and bus stops. In other words, a mix suitable for open highway use may not be the most suitable for all urban uses. The matter was not considered per se in this study but a supposition can be offered for mixes that must be designed to withstand high horizontal stresses. Flaky particles, which give a highly oriented structure, should be avoided and cubical or chunky particles, which tend to produce a more isotropic structure, should be preferred. The use of cubical or chunky shaped particles is therefore to be encouraged for most urban uses where there is a high incidence of horizontal stresses. As an alternative, or in addition, to this the use of a high viscosity

8.3.7 (cont.)

binder or a mix with a low "flow" value will improve performance under such conditions, provided sufficient cohesion is maintained.

8.3.8 Theoretical Representation

The dynamic response of asphalt concrete to a moving wheel load is characterised by the viscoelastic behaviour of the binder and the dimensional effects of the solid aggregate particles confined in a layer. Elastic theory is completely inadequate for predicting the response of this material in the surface course although this does not deny its value for study of the whole pavement structure.

8.3.9 Anomalies in Published Data

Other published studies of traffic compaction have described some surprising anomalies. These included a decrease in density over one period of trafficking and an observation of a greater density increase between the wheelpaths than in them. The suggestion that thermal cycling was the cause of the latter had the support of some laboratory evidence but no support from field evidence. A density increase between the wheelpaths can then only be attributable to the passage of some wheel loads in that area. A decrease in density, if it is accompanied by some distortion of the pavement, may be due to upthrust. Otherwise it seems unlikely to be anything other than a sampling error detecting inherent density variations in the material.

8.3.10 Practice

The study has shown that high stability mixes commonly used in New Zealand do undergo compaction by normal highway traffic. Subject to the foregoing considerations asphalt concrete mixes designed to current National Roads Board specifications, and used in the study, performed satisfactorily.

SUGGESTIONS FOR FUTURE RESEARCH

A number of areas for future research are suggested:

(i) The influence of aggregate shape and gradation on compaction and deformation properties of asphalt concrete.

(ii) Investigation to find an absolute property that is definitive of "stable state" for any incident conditions. Linear viscoelastic theory holds some promise in this respect.

(iii) Furtherance of the particle orientation analysis to increase the knowledge of the mechanism of deformation and compaction. Other methods such as special x-ray techniques may be more useful than that presented in this thesis.

(iv) A pavement temperature survey in New Zealand would have value in providing reliable data for the range and incidence of temperatures occurring in this country.

REFERENCES

1. McLEOD, N.W.: in Mix design methods for asphalt concrete, The Asphalt Institute Manual MS-2, 2nd ed., 1962, p.115.

2. McLEOD, N.W.: Influence of viscosity of asphalt cements on the compaction of paving mixtures, H.Res.Rec. n158, 1967, pp.76 ff.

3. NEVITT, H.G.: Compaction fundamentals, Proc. AAPT v26, 1957, pp.201-206.

4. CAMPEN, W.H., SMITH, J.R., ERIKSON, L.G., and MERTZ, L.R.: The effect of traffic on the density of bituminous paving mixtures, Proc. AAPT v30, 1961, pp.378-397.

5. SHAH, S.C.: Compaction of asphaltic concrete pavement with high intensity pneumatic roller Part II - Densification by traffic, Res.Rep. n19, Louisiania Dept. of Highways, 1965.

6. GRAHAM, M.D., BURNETT, W.C., THOMAS, J.J., and DIXON, W.C.: Pavement density - What influences it?, Proc. AAPT v34, 1965, p.286 ff.

7. PALMER, R.K. and THOMAS, J.J.: Pavement density - How it changes, Proc. AAPT v37, 1968, pp.542-583.

8. EPPS, J.A., GALLAWAY, R.M., and SCOTT, W.W.(Jr): Long-term compaction of asphalt concrete pavements, H.Res.Rec. n313, 1970, pp.79-91.

9. HEITHAUS, J.J. and JOHNSON, R.W.: A microviscometer study of road asphalt hardening in the field and laboratory, Proc. AAPT v27, 1958, pp.17 ff.

10. CAMPEN, W.H., SMITH, J.R., ERIKSON, L.G., and MERTZ, L.R.: The relationships between voids, surface area, film thickness and stability in bituminous paving mixtures, Proc. AAPT v28, 1959, pp.149-178.

11. GOODE, J.F. and LUFSEY, L.A.: Voids, permeability, film thickness vs. asphalt hardening, Proc. AAPT v34, 1965, p.430.

12. MACK, C.: Rheology of bituminous mixtures relative to the properties of asphalts, Proc. AAPT v13, 1942, p.194 ff.

13. q.v. McLEOD[61], and MACK[38], p.148.

14. MACK, C.: Physical properties of asphalts in thin films, Ind. & Eng. Chem., Pt.I, v49, 1957, pp.422-427.

15. WELBORN, J.Y., HALSTEAD, W.J., and OLSEN, R.E.: Relation of absolute viscosity of asphalt binders to stability of asphalt mixtures, Pub.Roads v32 n6, 1963, pp.145-152.

16. BAHRI GANDHARV RAJ, and RADER, L.F.: Effects of asphalt viscosity on physical properties of asphaltic concrete, H.Res.Rec. n67, 1965, p.59.

17. BENSON, F.J.: Effects of aggregate size, shape and surface texture on the properties of bituminous mixtures - A literature survey, HRB Spec.Rep. S.R.109, 1970, pp.12-22.

18. HVEEM, F.N.: Gradation of mineral aggregates for dense graded bituminous mixtures, Proc. AAPT v12, 1940, pp.315-330.

19. McNAUGHTON, M.F.: Discussion of paper by W.H. Campen, Proc. AAPT v12, 1940, pp.309-314.

20. HVEEM, F.N. and VALLERGA, B.A.: Density versus stability, Proc. AAPT v21, 1952, pp.237-254.

21. STANTON, T.E.Jr. and HVEEM, F.N.: Role of the laboratory in preliminary investigation and control of materials for low cost bituminous pavements, Proc. HRB v14, Pt.II, 1935, pp.14-54.

22. MOAVENZADEH, F. and GOETZ, W.H.: Aggregate degradation in bituminous mixtures, H.Res.Rec. n24, 1963, pp.106-137.

23. BENSON, J.R.: The grading of aggregates for bituminous construction, ASTM Symposium on Mineral Aggregates, 1948, pp.117-133.

24. GRIFFITH, J.M. and KALLAS, B.F.: Laboratory investigations of the influence of fine aggregates on properties of asphaltic concrete paving mixtures, Proc. HRB v37, 1958, pp.219-258.

25. GOODE, J.F. and LUFSEY, L.A.: A new graphical chart for evaluating aggregate gradations, Proc. AAPT v31, 1962, pp.176-192.

26. WILLIAMS, E.G. and GREGG, L.E.: Evaluation of sandstone as an aggregate in plant mix bituminous pavements, Proc. AAPT v22, 1953, pp.92-106.

27. CAMPEN, W.H., SMITH, J.R., ERIKSON, L.G., and MERTZ, L.R.: Factors that control asphalt requirements of bituminous paving mixtures and a method for determining the proper asphalt content, Proc. AAPT v32, 1963, pp.530-544.

28. CSATHY, T.I. et al.: State of the art: Skid resistance research, HRB Spec.Rep. S.R.95, 1968, pp.34-48.

29. HERRIN, M.H. and GOETZ, W.H.: Effect of aggregate shape on stability of bituminous mixtures, Proc. HRB v33, 1954, pp.293-308.

30. SHKLARSKY, E. and LIVNEH, M.: The use of gravels for bituminous paving mixtures, Proc. AAPT v33, 1964, pp.584-610.

31. LEFEBVRE, J.A.A.: Recent investigations of design of asphalt paving mixtures, Proc. AAPT v26, 1957, pp.321-394.

32. WEDDING, P.A. and GAYNOR, R.D.: The effects of using crushed gravel as the coarse and fine aggregate in dense graded bituminous mixtures, Proc. AAPT v30, 1961, pp.469-492.

33. FIELD, F.: The importance of percent crushed in coarse aggregates as applied to bituminous pavements, Proc. AAPT v27, 1958, p.294.

34. PUZINAUSKAS, V.P.: Influence of mineral aggregate structure in bituminous mixtures, H.Res.Rec. n51, 1964, pp.1-15.

35. LEES, G. and SALEHI, M.: Orientation of particles with special reference to bituminous paving materials, H.Res.Rec. n273, 1969, pp.63-75.

36. PFEIFFER, J.P.: The properties of asphaltic bitumen, Elsevier Pub. Co., 1950.

37. van der POEL, C.: Representation of rheological properties of bitumens over a wide range of temperatures and loading times, Proc. 2nd Int.Cong.Rheol., Oxford 1954, pp.331-7.

38. MACK, C.: Deformation mechanism and bearing strength of bituminous pavements, Proc. HRB v33, 1954, pp.138-166.

39. McLEOD, N.W.: A rational approach to the design of bituminous paving mixtures, Proc. AAPT v19, 1950, pp.82-224.

40. HEWITT, W.L. and SLATE, F.O.: The effects of the rheological properties of asphalt on strength characteristics of asphalt concrete, Proc. 2nd.Int.Conf.Struct.Des.Asph.Pvts 1967, pp.757-767.

41. NIJBOER, L.W.: Mechanical properties of asphalt materials and structural design of roads, Proc. HRB v33, 1954, pp.185-200.

42. GOETZ, W.H., McLAUGHLIN, J.F. and WOOD, L.E.: Load-deformation characteristics of bituminous mixtures under various conditions of loading, Proc. AAPT v26, 1957, pp.237-296.

43. PAGEN, C.A.: Rheological response of bituminous concrete, H.Res.Rec. n67, 1965, pp.1-26.

44. BLAND, D.R.: The theory of linear viscoelasticity, London, Pergamon Press, 1960.

45. PAPAZIAN, H.S.: The response of linear viscoelastic materials in the frequency domain with emphasis on asphaltic concrete, Proc.Int.Conf.on Struct.Des.Asph.Pvts, Ann Arbor, Michigan, 1962, pp.412-440.

46. YACKOVLEV, V. and BARENBERG, E.J.: Some aspects of the rheological behaviour of bituminous mixtures, H.Res.Rec. n273, 1969, pp.12-27.

47. HUANG, Y.H.: Deformation and volume change characteristics of a sand-asphalt mixture under constant direct and triaxial compressive stresses, H.Res.Rec. n178, 1967, pp.60-74.

49. DILLARD, J.H.: Comparison of density of Marshall specimens and pavement cores, Proc. AAPT v24, 1955, pp.178-232.

50. SYMPOSIUM ON ASPHALT PAVING MIXTURES: HRB Res.Rep. 7B, 1949.

51. METCALF, C.T.: Relation of densification to performance of small-scale asphaltic concrete test sections, HRB Bull. n234, 1959, pp.1-11.

52. McLEOD, N.W.: Relationships between density, bitumen content and voids properties of compacted bitumen paving mixtures, Proc. HRB v35, 1956, pp.327-404.

53. METCALF, C.T.: Use of Marshall stability test in asphalt paving mix design, HRB Bull. n234, 1959, pp.12-22.

54. SWANSON, R.C., NEMEC, J.Jr. and TONS, E.: Effect of asphalt viscosity on compaction of bituminous concrete, H.Res.Rec. n117, 1966, pp.23-53.

55. VOKAC, R.: Repeatability of Marshall test by analysis of factorial experiment data, Proc. AAPT v31, 1962, pp.533-562.

56. LEHMANN, H.L. and ADAM, V.: Application of Marshall method to hot mix design, ASTM Spec.Tech.Publn. n252, 1959, pp.22-40.

57. NEVITT, H.G.: Some sources of stability measurement variations, Proc. AAPT v28, 1959, pp.16-33b.

58. GOETZ, W.H.: Comparison of triaxial and Marshall test results, Proc. AAPT v20, 1951, pp.200ff.

59. McLAUGHLIN, J.F. and GOETZ, W.H.: Comparison of unconfined and Marshall test results, Proc. AAPT v21, 1952, pp.203ff.

60. SMITH, V.R.: Triaxial stability method for flexible pavement design, Proc. AAPT v18, 1949, pp.63ff.

61. McLEOD, N.W.: Application of triaxial testing to the design of bituminous pavements, ASTM Spec.Tech.Publn. n106, 1950.

62. KROKOSKY, E.M. and CHEN, J.P.: Viscoelastic analysis of the Marshall test, Proc. AAPT v33, 1967, pp.406-436.

63. TARAGIN, A.: Lateral placement of trucks on 2-lane highways and 4-lane divided highways, Pub.Roads v30, n3, 1958, pp.71-75.

64. NIVEN, D.: The rational design of flexible pavements, Unpublished thesis, University of Canterbury.

65. MARTIN, H.: Pressure distribution on the contact surface between tyre and road, Kraftfahrttechnisque Forschungsarbeiten, Heft 2, Berlin, 1936, 6.

66. TELLER, L.W. and BUCHANAN, J.A.: Determination of variation in unit pressure over the contact area of tires, Pub.Roads v18, n10, 1937, pp.195-198.

67. MARKWICK, A.H. and STARKS, H.J.: Stresses between tyre and road, Jnl.Instn.Civ.Engrs.(London) v16, 1941, pp.309-325.

68. FREITAG, D.R. and GREEN, A.J.: Distribution of stresses on an unyielding surface beneath a pneumatic tire, HRB Bull. n342, 1962, pp.14-23.

69. SAVKOOR, A.R.: Some aspects of friction and wear of tyres arising from deformations, slip and stresses at the ground contact, "Wear" v9, 1966, pp.66-78.

70. BONSE, R.P.H. and KUHN, S.H.: Dynamic forces exerted by moving vehicles on a road surface, HRB Bull. n233, 1959, pp.9-32.

71. LISTER, N.W. and NUNN, D.E.: Contact areas of commercial vehicle tyres, RRL Rep. LR172, 1968, 16pp.

72. MORRIS, S.A.H. and GALLOWAY, J.W.: Effect of road surface irregularities and structural variability on dynamic pressures under flexible roads, RRL Rep. LR215, 1968.

73. FREITAG, D.R., GREEN, A.J. and MURPHY, N.R.Jr.: Normal stresses at the tire-soil interface in yielding soils, H.Res.Rec. n74, 1965, pp.1-18.

74. YANDELL, W.O.: The effect of repeated rolling of elasto-plastic roads, Austral.Rd.Res. v3, n7, Sept. 1968, pp.14-20.

75. WHIFFIN, A.C. and MORRIS, S.A.H.: Piezoelectric gauge for measuring dynamic stresses under roads, The Engineer, London, n213, 1962, pp.741-746.

76. PEUTZ, M.G.F., JONES, A. and van KEMPEN, H.P.M.: Layered systems under normal surface loads H.Res.Rec. n228, 1968, pp.34-45, available as a computer program by the courtesy of Shell Oil Co. Ltd.

77. LISTER, N.W. and JONES, R.: The behaviour of flexible pavements under moving wheel loads, Proc. 2nd Int.Conf.Struct. Des.Asph.Pvts, Ann Arbor, Michigan, 1967, pp.1021-1035.

78. HEUKELOM, W. and KLOMP, A.J.G.: Consideration of calculated strains at various depths in connection with the stability of asphalt pavements, Proc. 2nd Int.Conf.Struct.Des.Asph.Pvts, Ann Arbor, 1967, pp.155-168.

79. ANDERSSON, O.: Rheological behaviour of asphalt pavings under traffic, Proc. 2nd Int.Conf.Struct.Des.Asph.Pvts, 1967, pp.479-486.

80. KLOMP, A.J.G. and NIESMAN, Th.W.: Observed and calculated strains at various depths in asphalt pavements, Proc. 2nd Int.Conf.Struct.Des.Asph.Pvts, Ann Arbor, 1967, pp.671-688.

81. GUSFELDT, K.H. and DEMPWOLFF, K.R.: Stress and strain measurements in experimental road sections under controlled loading conditions, Proc. 2nd Int.Conf.Struct.Des.Asph.Pvts., Ann Arbor, 1967, pp.663-669.

82. MORGAN, J.R. and SEARLE, G.K.: Strains in bituminous concrete pavements, Proc. ARRB v5, Pt.4, 1970, pp.218-228.

83. BROWN, S.F. and PELL, P.S.: An experimental investigation of the stresses, strains and deflections in a layered pavement structure subjected to dynamic loads, Proc. 2nd Int.Conf.Struct. Des.Asph.Pvts., Ann Arbor, 1967, pp.487-504.

84. DUNSTAN, D.G.: Temperature variations in a bituminous concrete surfacing at a site near Melbourne, Austral. Road.Res. v3, n3, 1967, pp.3-11.

85. KALLAS, B.F.: Asphalt pavement temperatures, H.Res.Rec. n150, 1966, p.1-11.

86. GALLOWAY, J.W.: Temperature durations at various depths in bituminous roads, R.R.L. Rep. LR138, 1968.

87. ARENA, P.J.: Service temperature study for asphaltic concrete, Res.Rep. 14, Lousiania Dept. Highways, 1964.

88. ROBERTS, P.K. and RUSSAM, K.: Road temperatures in the tropics, RRL Rep. LR29, 1966.

89. POLLARD, J.S.: The interrelation of atmospheric, binder and road surface temperatures, Proc. N.Z. Roading Symp. v2, 1967, pp.803-839.

90. McNAMARA, G.P.: Personal communication, 1970.

91. WELBORN, J.Y.: Asphalt hardening - Fact and fallacy, Public Roads v35, n12, 1970, pp.279-285.

92. GOODE, J.F. and OWINGS, E.P.: A laboratory-field study of hot asphalt concrete wearing course mixtures, Public Roads v31, n11, 1961, pp.221-228.

93. AASHO ROAD TEST REP. 5 "PAVEMENT RESEARCH": HRB Spec.Rep. SR61E, 1962.

94. WILLIMAN, A., MAJOR, N.G., FORD, I.L., SUGGATE, G.C. and NIVEN, D.: Christchurch-Lyttelton road pavement performance study: The experimental road and its instrumentation, Proc. 2nd Conf. ARRB, 1964, vII, pp.802-829.

95. GARRISON, W.A.: Three-year evaluation of Shell Avenue test road, H.Res.Rec. n117, 1966, pp.71-95.

96. McRAE, J.L. and FOSTER, C.R.: Theory of application of a gyratory test machine for hot mix bituminous pavement, ASTM Spec. Tech. Publn. n252, 1959, p.9ff.

97. BUSCHING, H.W. and GOETZ, W.H.: Use of a gyratory testing machine in evaluating bituminous mixtures, H.Res.Rec. n51, 1964, p.19.

98. MINOR, C.E.: Correlation of Hveem stabilometer and cohesiometer test results and kneading compactor densities with service records of bituminous pavements, ASTM Spec. Tech. Publn. 252, 1959, p.70.

99. GOETZ, W.H. and SCHAUB, J.H.: Triaxial testing of bituminous mixtures, ASTM Spec. Tech. Publn. n252, 1959, p.51.

100. CSANYI, L.H. and HON-PONG FUNG: Traffic simulator for checking mix behaviour, H.Res.Rec. n51, 1964, pp.67ff.

101. SPARKS, G.H., TAYLOR, H. and LEE, I.K.: Development and instrumentation of the model road test track at Sydney University, Proc. 4th Conf. ARRB, 1968.

102. TARRANT, A.G. and SLATER, R.: Road testing machines, Report of Road Res. Bd, U.K., 1936.

103. SPEER, T.L.: Laboratory traffic tests of miniature asphalt highways; second progress report, Proc. AAPT v31, 1962, pp.507-532.

104. AHLBERG, H.L. and BARENBURG, E.J.: The University of Illinois pavement test track - (A tool for evaluating highway pavements) H.Res.Rec. n13, 1963.

105. Koninklijke - Shell Laboratory, Amsterdam.

106. PAVEMENT RESEARCH CENTRE of the Division of Industrial Research, Washington State University publication, 1967.

107. GRAINGER, G.D.: Reconstruction of No. 3 road machine for pavement design studies, R.R.Lab note LN/506/GDG, 1964.

108. WILLIMAN, A. and PATERSON, W.D.O.: Pavement performance test track at University of Canterbury, N.Z. Road Research Newsletter n21, 1968, pp.2,3.

109. WILLIMAN, A. and PATERSON, W.D.O.: A track for the accelerated testing of highway pavements, N.Z. Engineering v26, n3, 1971, pp.73-77.

110. PIGNATARO, L.J.: Effect of test temperature on Marshall stability of asphaltic concrete mixtures, Proc. AAPT, v31, 1962, pp.563-597.

111. LEE, O-Y.: Evaluation of Marshall stability and flow values of asphaltic paving mixtures, H.Res.Rec., n273, 1969, pp.53-62.

112. JACKSON, G.P. and BRIEN, D.: Asphaltic concrete, Shell Int. Pet. Co. Ltd, London, 1962.

113. SELIG, E.T.: A review of stress and strain measurement in soil, Proc. Symp. on Soil-Structure Interaction, Univ. of Arizona, Tucson, Ar., pp.172-186, 1964.

114. PATERSON, W.D.O.: The measurement of pavement behaviour using an induction coil strain gauge, N.Z. Road Res. Bulletin, in press, July, 1972.

115. SCALA, A.J.: Personal Communication, Aug. 1971.

116. MORGAN, J.R.: Personal Communication, 1971.

117. DEISSLER, R.G. and EIAN, C.S.: Natl. Advisory Com. Aeronautics, RME52C05 (1952).

118. SCHOTTE, W.: A.I.Ch.E. Jnl. n6, 63, 1960.

119. HUDSON, W.R. and McCULLOUGH, B.F.: An extension of rigid pavement design methods, HRB Res. Record, n60, 1960, p.1.

120. DUNLOP (N.Z.) LTD: Personal Communication, April 1968.

121. SAITO: 9th IATC, Fisita, 1962.

122. WILLIAMSON, P.J.: The estimation of heat outputs for road heating installations, U.K.R.R.L. Rep. LR77, 1967.

123. McADAMS, W.H.: Heat transmission, McGraw Hill, 1954,

124. - See Literature quoted in Ref. No. 35.

125. HARRISON, P.W.: New technique for 3-dimensional fabric analysis of till and englacial debris containing particles from 3 to 40 mm size, Jour. Geology, v65, 1957, pp.98-105.

126. MIDDLETON, G.V.: The Tukey chi-square test, Jour. of Geology, v73, 1965, pp.547-549.

BIBLIOGRAPHY

The following is a short list of additional reading in selected areas:

Compaction and Accelerated Testing

ADAM, V., SHAH, S.C. and ARENA, P.J.(Jr): Compaction of asphaltic concrete pavement with high intensity pneumatic roller - Part I, Res.Rep. n10, Louisiana Dept. Highways, July 1963.

ADAM, V.: Compaction of bituminous concrete with self-propelled pneumatic tire rollers in Louisiania, Proc. 17th Ann.Conv. of SASHO, 1958.

BARNES, H.G. and WOODFORD, G.C.: Full-scale experiments with thin bituminous surfacings in Ayrshire, RRL Rep. LR176, 1968.

CARPENTER, C.A. and GOODE, J.F.: Circular track tests on low cost bituminous mixtures, Pub. Roads, v17 n4, 1936.

FROMM, H.J.: The compaction of asphaltic concrete on the road, Proc. AAPT, v33, 1964, pp.241-284.

FROMM,H.J. and PHANG, W.A.: The compaction of asphaltic concrete on the road - Part II, Proc. AAPT, v35, 1966, pp.529ff.

SHAH, S.C.: Asphaltic concrete pavement survey, Pub., Louisiania Dept. of Highways, April 1963.

SPEER, T.L.: Progress reports of laboratory traffic tests on miniature bituminous highways, Proc. AAPT, v29, 1960, p.316.

\- Accelerated testing of highway pavements, Iowa Engineering Experimental Stn., Private Communication.

\- Density studies of asphalt concrete, N.Y. State Dept. of Transportation, Res.Rep. 68-2, 1968.

Properties of Asphalt Concrete

BRIGHT, R., STEED, B., STEELE, J. and JUSTICE, A.: The effect of viscosity of asphalt on properties of bituminous wearing surface mixtures, Proc. AAPT, v35, 1966, pp.164-206.

JONES, G.M., DARTER, M.I. and LITTLEFIELD, G.: Thermal expansion-contraction of asphaltic concrete, Proc. AAPT, v37, 1968, pp.56-100.

KRCHMA, L.C. and GROENING, T.: Influence of pavement voids, asphalt content, and asphalt grade on asphalt performance, Proc. AAPT, v28, 1959, pp.34-52.

OLSEN, R.E.: Stability of bituminous pavements related to aggregate characteristics, Public Roads, v33 n3, 1964, pp.54-56.

SPEER, T.L. et al: Asphalt viscosity as related to pavement performance, Symposium Proc. AAPT, v32, 1963, p.236.

U.S. CORPS OF ENGINEERS: Investigation of the design and control of asphalt paving mixtures, Tech. Memo. No. 3-254.

Density Measurement

HUGHES, C.S. and RALSTON, H.H.: Field testing of a nuclear density device on bituminous concrete, Proc. AAPT, v32, 1963, p.106.

KIMBLE, F.W.: Field method of in-place density determination and control of hot mix asphaltic concrete, Proc. AAPT, v27, 1958, p.265.

LARSEN, O.: Development and use of high pressure apparatus for determining voids in compacted bituminous concrete mixtures, Proc. AAPT, v26, 1957, pp.403-436.

O'FLYNN, O.T.: Compaction control of bituminous concrete by air permeability measurement, Proc. 3rd Conf. ARRB, 1966, p.173.

Bituminous Materials

HOIBERG, A.J.: Bituminous materials - Asphalts, tars and pitches, Interscience Pub. N.Y., v1-3, 1964.

VAN DER POEL, C: A general system describing the viscoelastic properties of bitumens and its relation to routine test data, Jnl. Appl. Chem. 4, 1954, p.221.

Temperature

DICKINSON, E.J.: Recording the temperature environment of road surfacings, Aust. Road Res., v2 n5, 1965, p.63.

DICKINSON, E.J.: Temperature conditions in bituminous concrete surfacings at a site near Melbourne during a period of 3 years, ARRB Proc. v3 n9, 1969, pp.35-41.

DICKINSON, E.J.: Temperature conditions in bituminous surfacings at a site near Sydney during a period of one year, ARRB Proc. v3 n9, 1969, pp.42-48.

RUBIN, L.G.: Measuring temperature, Internat. Sc. & Tech., n25, 1964, pp.74-8, 80, 83-88.

TROTT, J.J.: An apparatus for recording the duration of various temperatures in roads, Roads & Road Constr., v41, 1963, p.491.

Miscellaneous

DUNN, R.C.M.: The effect of edgemarking on vehicle placement, Proc. N.Z. Roading Symposium, v1, 1967, pp.209-233.

STEWART, M.: A new method of producing texture prints of road surfaces, RRL, LN/610/MS, 1964.

GLOSSARY OF SPECIAL TERMS

Air Voids Unit (a.v.u.): equivalent to 1% air voids content by volume.

Apparent Dynamic Modulus (A.D.M.): The ratio of tyre contact pressure to peak vertical compressive average strain (MN/m^2), cf. section 7.2.2, p.177.

Average Strain: The average deformation per unit length in a non-homogeneous mixture.

Bearing Capacity: A function relating Marshall stability and flow values to load-bearing characteristics, cf. section 2.3.2, p.29; defined by Metcalf[53].

Compaction: A system of imposed stresses that causes densification and restructuring of a material; frequently causing a greater increase in strength than in density of the material, cf. section 1.2.1, p.3.

Densification: An increase in the mass per unit volume of a material with essentially no restructuring.

Normalised Strain: The ratio of strain to applied contact pressure, cf. section 7.2.2, p.175; defined by Brown and Pell[83].

Squareness Function: A term defined to indicate the attainment of stable state conditions as a function of traffic distribution across the path of the Pavement Testing Vehicle, cf. section 6.1.2, p.103.

Stable State Conditions: The material conditions at which asphalt concrete resists all further compaction by traffic at load and temperature conditions less than or equal to the Testing Conditions being considered, cf. section 5.1.1, p.69.

Stage: A stage during trafficking on the Track representing one particular combination of tyre contact pressure and surface course temperature, i.e. one Testing Condition, cf. section 5.4.3, p.85.

Testing Condition: see "Stage".